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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

THE EIGHT-MILE CASCADE TUNNEL GREAT NORTHERN RAILWAY

A SYMPOSIUM

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SYNOPSIS

This Symposium constitutes the final professional record of the construction of the 7.79-mile Cascade Tunnel by the Great Northern Railway Company. It is presented in three parts, each part being written by an official of the Great Northern Railway Company, or of A. Guthrie & Co., Inc., the contractors on this work.

Part I, by Mr. Kerr, is an outline of the history of the Great Northern Railway line over the Cascade Mountains in the State of Washington. It also contains a discussion of the reasons leading to the improvement of this mountain crossing.

Part II, by Colonel Mears, contains a description of the surveying methods and instruments used in constructing the tunnel. It presents the general plan of attack, the construction standards, and a description of the line improvement on the east and west slopes of the Cascade Mountains; and it gives a comparison of the physical characteristics of the new and old routes.

Part III, by Mr. Baxter, with two appendices by H. J. King, M. Am. Soc. C. E., is a study of the plans made for the tunnel and the detailed problems affecting methods of construction.

PART I.—PRELIMINARY STUDIES AND RESULTS OF IMPROVING CASCADE CROSSING

BY D. J. KERR,[1] M. AM. SOC. C. E.

The Great Northern Railway, while not the most recently built transcontinental line, was only opened through to the Pacific Coast in January, 1893. It was in December, 1889, that John F. Stevens, Past-President and Hon. M. Am. Soc. C. E., explored the Marias Pass in the Rocky Mountains and thus opened up a short and low summit route for a line not far south of the Canadian border. Mr. Stevens did not have the satisfaction of staying to complete the location survey over the Rockies because his success there resulted in the late James J. Hill, F. Am. Soc. C. E., sending him to the Cascade Range to determine the route of the Great Northern Railway across that barrier.

In 1890, there was little information available about the Cascade Mountain passes in Northern Washington, and the slopes were heavily timbered. The route to the sea by way of the Fraser River in British Columbia and by way of the Columbia River along the boundary between Washington and Oregon had been known to white men for nearly a hundred years. The Northern Pacific transcontinental line, opened in 1887, followed the Valley of the Yakima River (see Fig. 1) from its confluence with the Columbia, near Pasco, Wash., and crossed the Cascade Range through the Stampede Tunnel to the head-waters of Green River, and thence down the Green River Valley to the Pacific Coast.

Mr. Hill, as he had done across Dakota and Montana, wanted the line across the State of Washington to open up new territory, and Mr. Stevens proceeded to make a thorough examination of the Cascade Range between the Northern Pacific crossing and the Canadian boundary, a distance "as the crow flies" of 125 miles. A reference to the map, Fig. 1, will make the problem and the description clearer.

The result of Mr. Stevens' work was the selection of a pass which connected the head-waters of Nason Creek, a tributary of the Wenatchee River, with Tye River, a tributary of the Skykomish River. The Wenatchee River joins the Columbia River near Wenatchee, Wash., while the Skykomish flows into the Snohomish which enters the sea at Everett, Wash. The route, therefore, was fixed from Spokane, Wash., *via* Wenatchee, the new pass (since known as Stevens Pass), and Everett; thence, 35 miles along the seashore, to Seattle, Wash.

The pass had an elevation of 4 059 ft. The approach from the east was through the narrow Tumwater Canyon of the Wenatchee River and on the west side the slopes fell away so rapidly as to necessitate considerable development to obtain a reasonable grade line. Mr. Stevens selected 2.2% as the maximum grade of the permanent line which was to be located down hill on each side from the portals of a projected tunnel, 2.7 miles long. The summit was to be at the east portal of this tunnel at an elevation of 3 383 ft.

[1] Asst. to Vice-Pres., Operating Dept., G. N. Ry., St. Paul, Minn.

The Great Northern Railway Company had no land grant west of Minnesota, and it was necessary to develop traffic quickly to justify the construction of the extension. The opening of the through line could not be delayed awaiting the completion of a tunnel nearly three miles long and, therefore, a switchback route with 4% grades was located over the mountain across the

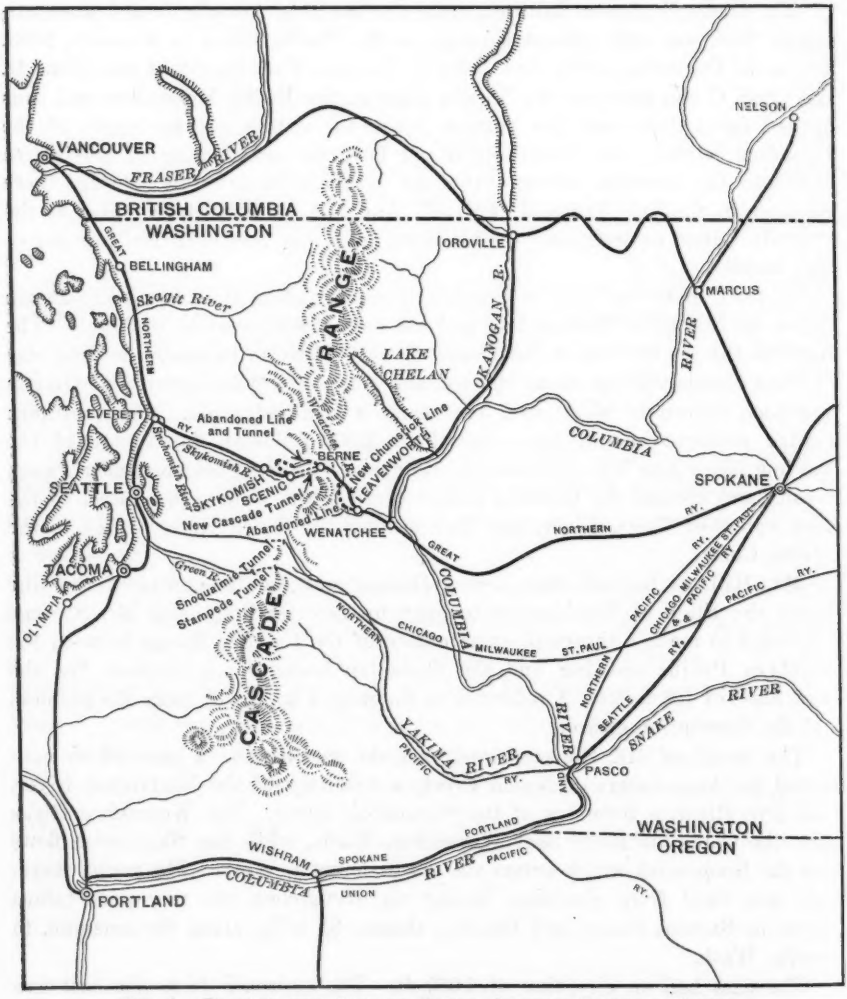

FIG. 1.—PLAN OF THE CASCADE RANGE, SHOWING RAILROAD ROUTES.

open pass and connecting with the permanent line near the portals. This switchback route served to handle the through traffic of the Railway Company from January, 1893, until December, 1900, when the 2.7-mile Cascade Tunnel was completed. As a result the summit elevation was reduced from 4 059 to 3 383 ft., and all grades heavier than 2.2% were eliminated. The profile of the line from Wenatchee to Skykomish, Wash., is shown in Fig. 2.

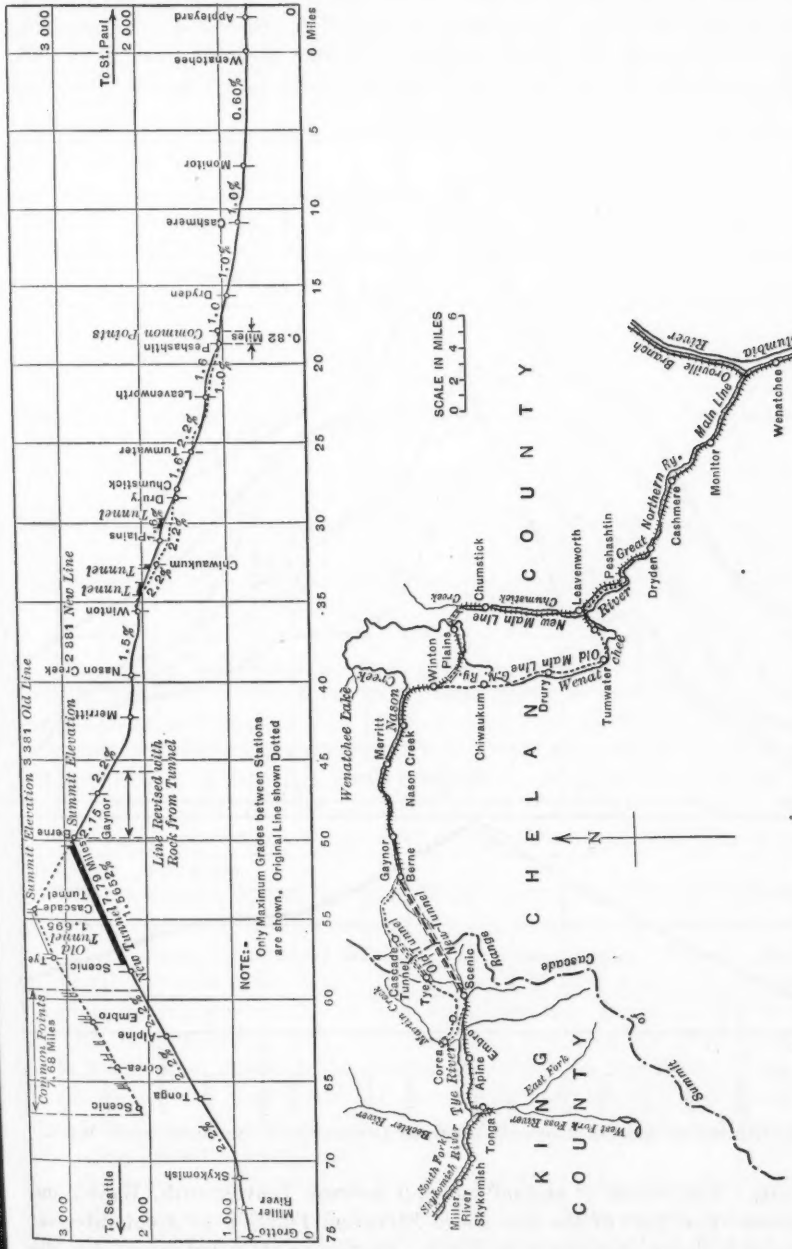

FIG. 2.—MAP AND PROFILE, SHOWING OLD AND NEW MAIN LINES BETWEEN APPLEYARD AND SKYKOMISH, WASH.

There was little evidence of snowslide trouble on the heavily timbered slopes along which the line located by Mr. Stevens lay, but the destruction of timber above the railroad by fire soon altered the situation. Snowslides of increasing extent developed and the need of protection became apparent as early as 1903. Nearly every year thereafter it was found necessary to extend the snow-shed

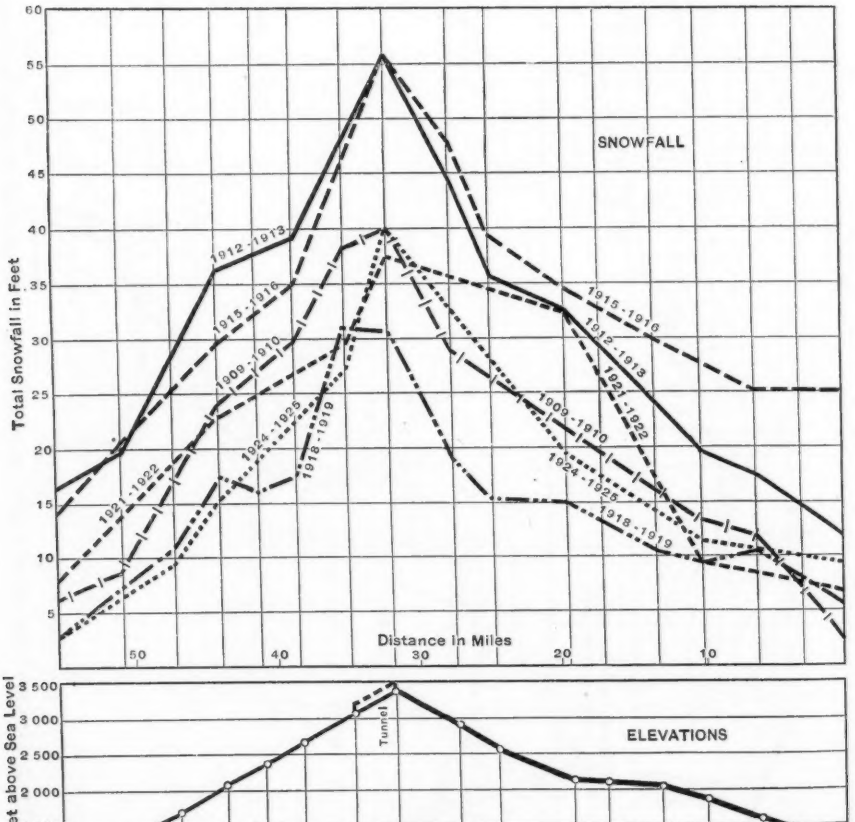

FIG. 3.—RECORD OF ANNUAL SNOWFALL BETWEEN LEAVENWORTH AND SKYKOMISH, WASH.

protection. The record of annual snowfall between Leavenworth, Wash., and Skykomish (that part of the line above Elevation 1 000) is of great interest; this record is shown graphically in Fig. 3. Particular attention is called to the heavy snowfalls during the winters of 1909-10, 1912-13, and 1915-16.

The sections of the line exposed to snowslide menace were rather limited and consisted of two districts—one on each slope. On the east slope the trouble was confined to the line through the Tumwater Canyon between Leavenworth and Winton, Wash., a distance of 13 miles; on the west slope the trouble zone extended from the west portal of the old tunnel to Scenic, Wash., a distance of 12 miles. The rapidity with which this menace developed and extended is shown best in Table 1 which gives the number of linear feet of track protected at the end of each year.

TABLE 1.—LINEAR FEET OF LINE PROTECTED AT THE CLOSE OF EACH YEAR

Year	TUMWATER CANYON, LEAVENWORTH TO WINTON	CASCADE MOUNTAIN CROSSING, MERRITT TO SCENIC	
	Sheds	Sheds	Tunnels
1903	.....	3 894	15 595
1905	.....	4 833	15 595
1907	.....	7 044	15 595
1909	.....	7 638	15 595
1911	1 429	14 766	15 595
1913	2 022	25 877	15 595
1915	2 022	26 221	16 816
1917	8 401	32 033	19 332
1919	8 401	32 412	19 332
1921	8 401	32 420	19 332
1923	8 401	32 313	19 332
1925	8 401	32 322	19 332

The earlier snow-sheds constructed before timber and labor costs had advanced as they have in recent years, were built for about $70 per lin. ft. for single-track and $83 per lin. ft. for double-track sheds. Those constructed with concrete back-walls and timber roofs cost $175 per lin. ft. of double-track shed. Many of the roof timbers were 24 in. by 12 in. by 42 ft., and such timbers are now high in price and increasingly difficult to obtain. At present prices, a double-track concrete back-wall shed, with a timber roof, costs almost $210 per lin. ft.

The increasing cost of replacement and the constantly increasing length of line requiring protection caused considerable thought to be given to the finding of a route that would avoid the snowslide areas entirely. Studies were made from time to time by the late A. H. Hogeland, M. Am. Soc. C. E., and by O. S. Bowen, M. Am. Soc. C. E. A most complete study of the problem was made by Mr. E. J. Beard in 1917, which not only compared the relative economies of a long tunnel line with the existing line from the standpoint of the cost of maintenance and construction of snow-sheds, but also from the standpoint of the transportation savings from the shorter, lower summit and easier grade line, coupled with electrification.

The conclusions reached by Mr. Beard in 1917 are summarized, as follows:

"That the present line in Tumwater Canyon, Leavenworth to near Winton, and from Merritt to Skykomish, be abandoned, substituting therefor changes in line by which all snow-sheds are avoided, providing either a 1.4% or a 1%

ruling grade and involving either a 14½ or a 17¼-mile tunnel, as chosen, and estimated to cost, including the changes, electrification, Leavenworth to Delta, necessary equipment, and 100-lb. rails, $25 000 000 or $27 000 000.

"This, because the present line, if retained, will have to be electrified some day, when the cost, plus the great expense that must be incurred account of snowsheds, existing and additional, would be $21 000 000, an investment that would make the property for years less profitable than it is at present; *per contra* the undertaking first above will, upon completion, increase net corporate income over that with the present line, electrified, about $350 000 per year, and the advantageous difference will increase each year.

"That neither the present line, steam operated or electrified, nor the undertaking first above should be double-tracked, because the surplus train capacity of either is enough for the increase in traffic of so many years in the future that no money whatever should be spent for, or in anticipation of, double-tracking.

"The present snow-sheds, with the minimum of additions that circumstances will permit—to be built of timber—be maintained at the least practicable cost until the undertaking first above is completed.

"That every effort be made and consistent expense be incurred to complete the undertaking first above in from 4½ to 5 years instead of the estimated 6½ to 7½ years, thereby reducing the estimates above by from $1 500 000 to $2 000 000."

Unfortunately, the intervention of the World War forced the project into the background and at the conclusion of the war neither the traffic nor the financial condition warranted the consideration of a large project. The full effect of the Panama Canal on trans-continental rail traffic was only felt as ship tonnage became plentiful in 1919 and 1920.

Discussion of the project was revived in 1921, and new estimates were made from that time until 1925. These were based largely on the 14-mile tunnel line recommended by Mr. Beard. By that time costs had changed materially and it was evident that the project would involve an expenditure in excess of $30 000 000. Such an outlay could not be justified economically at that time and, having reached the conclusion that a long tunnel would not be built, the Operating Department of the Railway Company began the improvement of operation over the existing line.

In earlier days trains were moved eastbound from Skykomish by coal-burning locomotives with the result that the atmosphere in the old tunnel (which was on a 1.69% grade against eastbound traffic) was intolerable. In 1909, a hydro-electric development was made in the Tumwater Canyon, a short distance above Leavenworth, and plans were begun for electrifying the tunnel.

The Tumwater Plant consisted of three 2 500-kw. units and power was transmitted to the tunnel at 33 000-volts, 3-phase, 25 cycles. The overhead trolley installation consisted of two 4-0 wires, carrying 6 600 volts, and extended about 3½ miles from Cascade Tunnel Station through the tunnel to Tye, Wash. Four 3-phase locomotives were purchased which had constant speed of 15 miles per hour, that being the only running speed available. The plan of operation was to have a train of about 2 000 tons leave Skykomish with a steam road engine and two steam pushers. At Tye (the west portal of the old tunnel), the train was cut in two and, with the road engine, was taken through to Cascade Tunnel Station (the east portal of the old tunnel) by electric locomotives.

The first move made in 1925 to improve operation was to re-arrange the electric locomotives so that the traction motors could be connected in cascade giving the locomotive when so connected a speed of 7.5 miles per hour instead of 15 miles per hour. This change permitted taking passenger trains through the tunnel at 15 miles per hour as before and freight trains in one section at 7.5 miles per hour. In this way the full power available at Tumwater (the only source) was used, and the tractive effort of all four locomotives was made useful for freight trains.

The success of this change prompted the Company to consider extending the electrification down the west slope to Skykomish, with the idea of using electric pushers throughout. The plan was to obtain two electric locomotives which, with steam engine working, would move a train from Skykomish to Tye on a 2.2% grade and which could haul the train and the steam road engine (not working) from Tye to the Cascade Tunnel through the old tunnel on the 1.69% grade.

The limited supply of power available at the Railway Company's Tumwater Plant and the need of variable-speed locomotives to work with steam road engines resulted in the selection of the motor generator type of locomotive with 11 000-volt, single-phase, 25-cycle trolley, and 600-volt, direct-current traction motors. Two such double-cab locomotives (each having a 1 500-kw. motor generator set) were ordered in 1925, and work had already been started on the electrification of the old line, when serious consideration was given again to the long tunnel plan.

The success of the operation of freight trains in one section through the old tunnel, the improvement in steam operation (which had permitted running freight trains through from Seattle and Everett to Wenatchee in less than the 16-hour crew limit), and the failure of the through freight traffic to show a material increase, made it apparent that savings in transportation expense alone could not justify any long tunnel line and that if such a project were to be warranted economically it must be the shortest tunnel which would permit the abandonment of all the line on the west slope subject to the snowslide menace.

A study made by Frederick Mears, M. Am. Soc. C. E., showed that this could be accomplished by a tunnel 7¾ miles long, having its east portal at Berne, Wash., and its west portal at Scenic. In contrast to the 14 or 17-mile tunnels recommended by Mr. Beard, this plan did not eliminate the 2.2% grade, but left in use the five miles of such grade against westbound traffic between Merritt, Wash., and Berne and eleven miles against eastbound traffic between Skykomish and Scenic.

Mr. Stevens was called upon to go over the ground that was so familiar to him and as a result of an exhaustive study in the summer of 1925, he urged the immediate approval of the 8-mile tunnel line as the only project that could be justified. He pointed out that there was much unwarranted prejudice against pusher districts and that with electrification the former objection to sections of 2.2% grade was largely eliminated, especially when such grades were comparatively short. Accordingly, the Board of Directors of the Railway

Company reached the conclusion in November, 1925, that the 7.79-mile tunnel should be built.

As a distinct and separate project on the east side of the mountains, a new line (the Chumstick Line) was projected from Peshastin (near Leavenworth) to Winton, Wash., a distance of 17 miles. It followed the open Chumstick Valley, eliminated all 2.2% grades east of Merritt, and provided a line with 3° maximum curvature, entirely free from the snowslide menace to which the old line in the Tumwater Canyon was exposed.

In addition to the major improvement made by the construction of the Chumstick Line on the east slope, numerous short sections of the old line between the west end of the Chumstick Line at Winton and the east portal of the new tunnel at Berne, were improved in alignment and grade.

When the decision to build the new tunnel was reached in November, 1925, the age of the existing snow-sheds made it extremely important, if heavy renewals were to be avoided, that the new route be made ready for use before the snowslide season of 1928-29. To avoid delay, the firm of A. Guthrie & Co., Inc., of St. Paul, Minn., was instructed to proceed with the work on a fixed fee basis—a plan which later was extended to cover the construction of the Chumstick Line.

The need of an adequate supply of power for the driving of the new tunnel made it advisable to turn over the Company's power station at Tumwater to the Puget Sound Power and Light Company and to contract with that Company for power to be supplied from its widespread system. Having such power available brought with it the decision to electrify the entire district from Wenatchee to Skykomish (75 miles).

These four pieces of work (1) the 7.79-mile Cascade Tunnel; (2) the Chumstick Line; (3) line revisions between Winton and Berne; and (4) the electrification from Wenatchee to Skykomish, constituted a complete program for the improvement of the Cascade crossing.

The first, in addition to the improved alignment and lower summit obtained, permitted the abandonment of all the line on the west slope subject to snowslides; the second, which also improved alignment and grade, resulted in the discontinuance of operation over all that part of the line on the east slope subject to slides; the third resulted in a marked improvement in grade and curvature; and the fourth was a necessary step following the decision to build the long tunnel and to avoid too frequent locomotive changing.

The completion of the program has put the Cascade crossing on an equal footing with the remainder of the main line in character, freedom from interruption, and freight-train tonnage that may be handled.

A comparison of the old and new line from the east end of the Chumstick revision, at Peshastin, to the west end of the tunnel revision at Scenic, is given in Table 2.

The electrification program, together with the change in line, grade, and rise and fall indicated in Table 2, has permitted the operation of 5 000-ton freight trains eastbound out of Skykomish, using a three-cab road locomotive and a two-cab helper. These helpers are operated generally between Skykomish and Merritt—the foot of the 2.2% grade on the east slope—thus avoiding

any stop at the summit at Berne. Westbound, in ordinary operation, the three-cab locomotive takes a 3 500-ton train from Wenatchee to Merritt, where a one or two-cab helper, whichever may be available, is used to Berne and remains in the train on the down-hill slope to Skykomish. As compared with the steam operation prior to any electrification on the slope west of the old tunnel, the train tonnage eastbound has been increased from 2 400 to 5 000. The average time for a freight train between Skykomish and Berne has been reduced 3½ hours and the gross ton-miles per train hour between Skykomish and Wenatchee have been increased from 11 520 to 70 000. The traffic on this part of the line is prevailingly eastward, substantially in the proportion of 5 000 to 3 000 tons.

TABLE 2.—COMPARISON OF OLD AND NEW LINES

	Old line.	New line.	Favorable to new line.
Length, in miles	49.32	40.90	8.88
Length of tangent track, in miles	26.86	27.66	0.80
Length of curved track, in miles	22.92	13.24	9.68
Total rise eastbound, in feet	1 353.70	898.92	454.78
Total rise westbound, in feet	2 342.15	1 887.37	454.78
Summit elevation, in feet	3 382.00	2 881.00	501.00
Length of snowsheds, in feet	40 153.00	841.00	39 312.00
Length of tunnels, in feet	19 620.50	48 600.40	28 979.90*
Maximum degree of curvature	10° 00′	10° 00′	.....
Total degree of curvature	5 617° 34′	2 059° 06′	3 558° 28′

* Unfavorable to the new line.

The various steps in the improvement of the railroad between Wenatchee and Skykomish have so overlapped that it is impossible to divide the savings resulting from the entire project into the part due to electrification and the part due to the improved alignment, lower summit, shorter mileage, etc. These savings in the aggregate are not more than sufficient to pay interest on the entire investment. The Railway Company must look for the justification of the project to the benefit which the system as a whole has received through the elimination of a weak link in its transportation chain, with consequent increased traffic resulting from service that is absolutely dependable, quicker, and, in the case of passenger travel, extremely comfortable and pleasant.

PART II.—SURVEYS, CONSTRUCTION METHODS, AND A COMPARISON OF ROUTES

By Frederick Mears,[2] M. Am. Soc. C. E.

Surveying parties were put into the field on the west slopes of the Cascades in September, 1925, to locate a line that would meet the general requirements of the problem, which called for reduction in curvature, saving in distance, and permanent protection of the line against snowslides. From a study of the United States Geological Survey maps of the Cascade Range, and other engineering data available, it was apparent that a tunnel approximately eight miles in length would meet these requirments, with east and west portal near Berne and Scenic, respectivly.

The line adopted provided for a single-track tunnel, 7.79 miles in length, on tangent, with 0.74 mile of new line approaching from the east and 1.54 miles from the west.

West Slope.—The locating party on the west slope made a detailed survey of the country in the Tye River Valley above Scenic, to determine suitable portal sites and proper gradients for the west approach, two portal sites in particular being developed, as shown by Fig. 4. Close topography was taken and soil and rock conditions were noted. The existence of a 0.99% grade on the main line, throughout the length of the old Scenic passing track, made it possible to raise the grade of the new line without exceeding the maximum 2.2% grade, so that 1½ miles of new embankment could be constructed on the floor of the Tye River Valley leading up to the face of the mountain.

The Cascade Range is in a region of high precipitation, particularly in the fall and winter months. Several hot springs were known to exist on the mountain slopes above Scenic, and every precaution had to be taken to locate the portal and approaches to avoid menace from snowslides, as well as to secure proper drainage.

The south slope of a ridge, in a bend of the Tye River, affording sufficient cover to "portal under" was finally selected as the west portal site, as shown in Fig. 5. The approaches were free from snowslides, and the Tye River provided good drainage for surface and underground water. It was known that the westerly ½ mile of the new tunnel would lie in a somewhat broken formation, requiring heavy timbering while in course of construction, but it was impossible to better this ground condition and, at the same time, meet other requirements.

East Slope.—On the east side of the Cascade Range, a portal site was located, as shown in Fig. 6, which met every requirement. A solid rock face of a mountain slope was selected for the portal, which could be reached from the operated line by the construction of less than a mile of new line, on good grade and alignment. This site was free from snowslides or drainage problems.

[2] Asst. Chf. Engr., G. N. Ry., Seattle, Wash.

Open Traverse.—As soon as the east and west portal sites were decided upon, a preliminary survey was made to connect the portals. This was an open traverse, about 10 miles long, following the operated line. (See Fig. 4.) The traverse showed that the tunnel would be on approximately a 1.6% grade, descending from east to west, and that the open approaches, at either end of the tunnel, could be built on a 1.5% grade, to provide for passing tracks of suitable length near the portals.

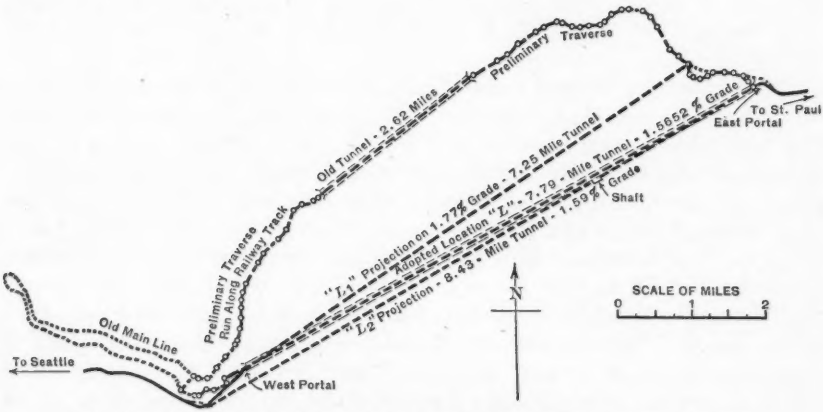

FIG. 4.—MAP SHOWING PRELIMINARY SURVEYS FOR NEW TUNNEL THROUGH THE CASCADE RANGE.

The preliminary traverse was extended up the valley of Mill Creek, and detailed surveys were made which developed that an intermediate shaft site could be located in the Mill Creek Valley, which would have the proper relation to the east and west portals and to the sub-grade of the proposed tunnel.

The floor of the valley of Mill Creek, where it is crossed by the axis of the tunnel (see Fig 7), was found to be about 622 ft. above the proposed tunnel sub-grade, at a point 2.41 miles west of the east portal. A shaft site, therefore, was laid out on the west slope of the valley, which will be referred to later. The relative location of this shaft is indicated on Fig. 8.

Second Survey.—Before establishing the tunnel tangent on the ground, a second survey was made, using more precise methods, but following very closely the first open traverse.

The azimuth of the long tunnel, calculated from this second traverse, was thrown across the four mountain ridges intervening between the portals. A light mountain type of transit was used in this field work and the error of closure between this first location of the tunnel axis and the open traverse, which followed the heavy curves of the operated line, was about 6 ft. The line thrown across the mountain tops was accepted as the preliminary tunnel axis, and the 8 by 9-ft. pioneer tunnel at the west portal, and the 10 by 10-ft. center heading at the east portal, were started by reference to it.

PRECISE SURVEYS OF THE LONG TUNNEL

The field work for the "definite location" of the New Cascade Tunnel, including the precise location of the intermediate shaft, in the rugged moun-

tains of the Cascade Range, was an interesting problem, in which the following fundamental steps were involved:

1.—Alignment.—The establishment of the tunnel axis across the mountain tops between portals, and the accurate projection of this line into the Valley of Mill Creek at the shaft site.

2.—Distance.—The measurement of the tunnel axis and the determination of the exact position of the Mill Creek shaft with respect to the east and west portals.

3.—Grade.—The determination of the difference in elevation between the east portal and the west portal and the mouth of the Mill Creek shaft.

Establishing the Tunnel Axis.—The first location of the tunnel axis across the mountains was made in December, 1925, using light, mountain-type transits. This was followed very shortly by a second survey conducted in a manner to secure greater accuracy. Precise transits of a larger type were used on this second survey, which were equipped with 11-in. telescopes of 28 power and horizontal plates, 7 in. in diameter, graduated to 10″ of arc. Several lines were run with these transits. The first operation showed that the preliminary tunnel axis established by the mountain type of transit was not a truly straight line; it contained a small constant error in the same direction, causing the line to follow a curve, with the east and west portals of the tunnel at either end. A "chord" to this curve, extending from portal to portal, was then calculated, and the offset distance determined for the two intermediate set-up points, at "Cowboy" Station and at "Big Chief" Station. (See Fig. 9.)

These two stations were then used to form a base line, from which the axis of the tunnel was produced toward the east portal and toward the west portal. Several trials were necessary before determining the exact points on Cowboy and Big Chief Mountains.

When the line had been refined by the precise transits to a degree such that it was considered safe to construct concrete monuments at the several observation points (seven in number, see Fig. 9), a high-power telescope direction instrument was used in the final adjustment of the line. Fig. 10 illustrates the type of instruments used for various parts of this work. As an ordinary tripod would not give the required stability, the precise transit was mounted on a 5-ft. section of 6-in. iron pipe, set vertically in concrete at each of the observation stations; it was equipped with a 22-in. telescope with two eye-pieces, one of 30 power and one of 45 power. The plate or horizontal circle has a diameter of 10 in. which, by use of a micrometer attachment, can be read directly to 2″ of arc. It is provided with a precise striding level, the graduations of which have a value of 5″ of arc. The instrument, without the bronze base on which it rests, weighs 41.5 lb., and with the base, 78.5 lb.; the detail of mounting, at the several observation stations, is shown in Fig. 11. About 18 in. of the lower end of the pipe was embedded in concrete, and openings, 3 in. in width and 7 in. in height, were cut in the pipe, just above the surface of the concrete, to permit the use of a plumb-bob in centering the instrument over the "point". Brass tacks, fixed in wooden hubs, were used at first to mark the points, but when the line was finally adjusted, the wooden hubs were replaced with brass plates set in concrete, on which the exact "point" was marked by a punch.

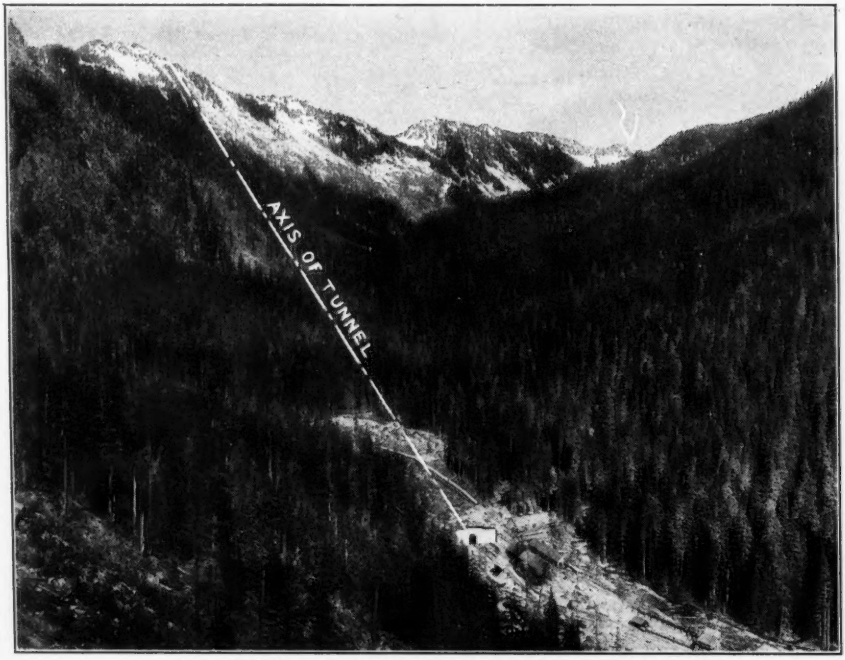

FIG. 5.—WEST PORTAL OF THE NEW CASCADE TUNNEL.

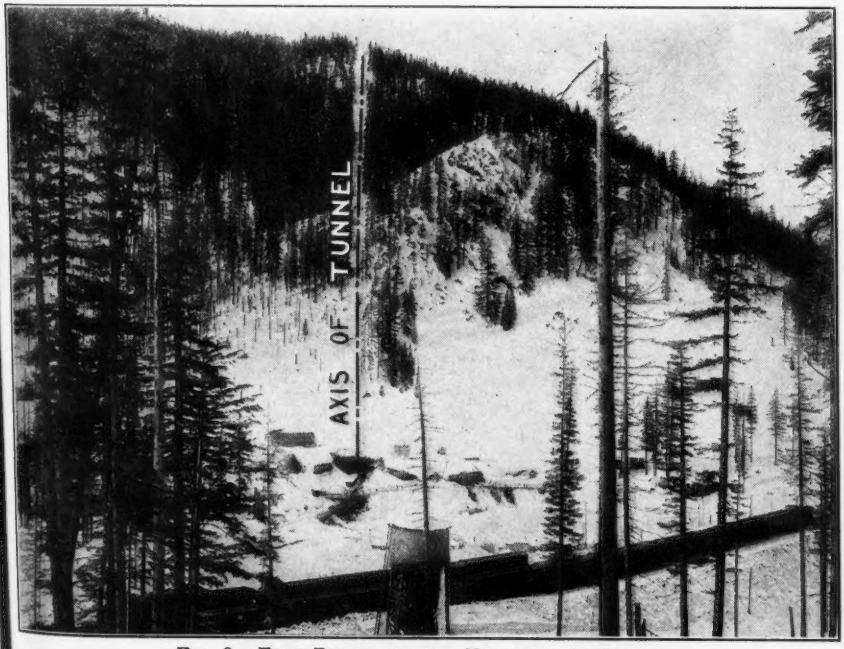

FIG. 6.—EAST PORTAL OF THE NEW CASCADE TUNNEL.

FIG. 7.—VIEW SHOWING FLOOR OF MILL CREEK VALLEY IN THE DIRECTION OF THE CASCADE TUNNEL.

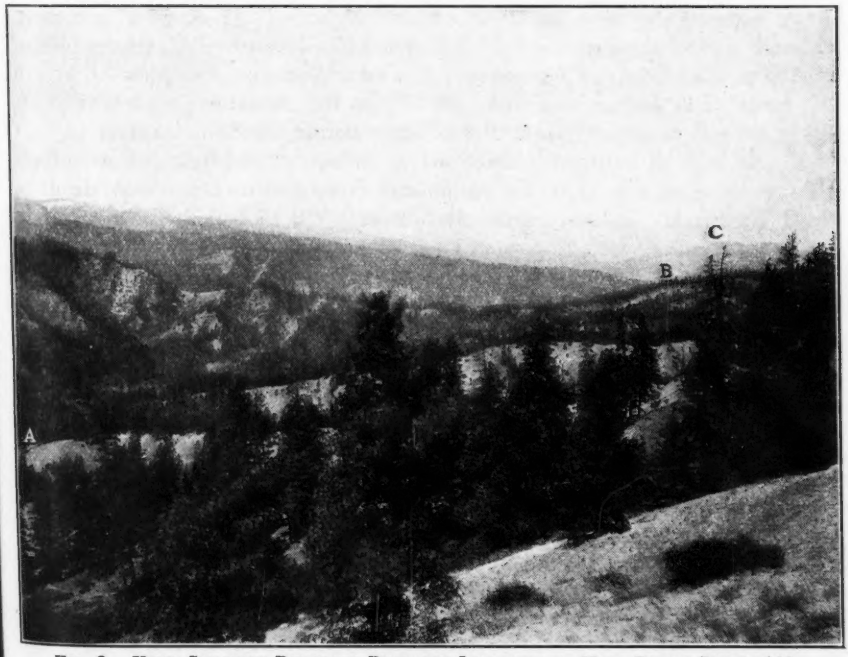

FIG. 8.—VIEW SHOWING RELATION BETWEEN LOCATION OF MILL CREEK SHAFT (A), WINTON TUNNEL (B), AND BIG CHIEF MOUNTAIN (C).

An inspection of Fig. 9 will show the observation stations established beyond the two portals, from which the tunnel tangent was produced underground to control all underground workings. It will also show the two intermediate observation stations which were constructed on the east and west slopes of Mill Creek Valley in order to overcome the great difference in elevation between Big Chief and the top of the Mill Creek shaft, in the location of the shaft site.

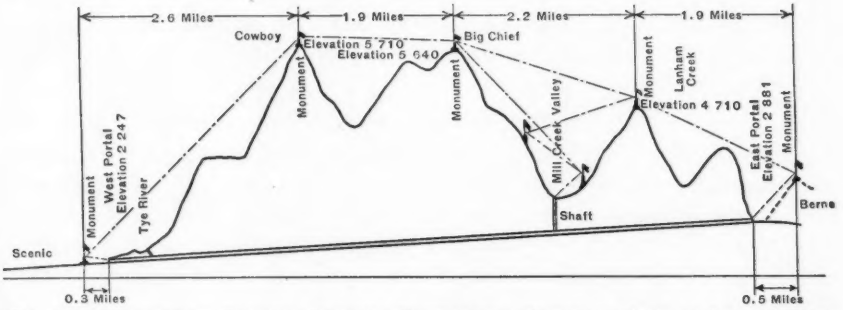

FIG. 9.—LONGITUDINAL SECTION, SHOWING OBSERVATION POINTS ON AXIS OF LONG TUNNEL, CASCADE RANGE.

Projection of Tunnel Axis from Top of Mill Creek Shaft.—The tunnel axis established on the surface of Mill Creek Valley, across the mouth of the shaft, was transferred to the bottom of the shaft in the following manner: The shaft was 8 by 24 ft., inside dimensions, with its long axis parallel to the tunnel tangent. Two piano wires, size No. 20, were suspended down the shaft 622 ft. to sub-grade, to which were attached two 60-lb. plumb-bobs in tubs of water to reduce pendulum motion. These wires were spaced 19 ft. apart.

The suspension of the plumb wires and their alignment at the top of the shaft was accomplished by means of two reels illustrated in Fig. 12. Each of these reels rested on two heavy timbers placed across the shaft collar. The piano wire was coiled in the groove of the drum marked "Hardwood Reel", and a small weight was attached to the free end of the wire and allowed to hang a short distance into the shaft, the wire being made to pass in the V-groove of the rod marked, "Rod 1 in. rd. x 1 ft. 4 in." Inspection of the diagram will show that the hanging wire can be seen through the frame from either end, so that the frame could be shifted into position by sighting on the wire with a surface transit, set up in proper relation to the tunnel axis.

The final adjustment was made by means of the grooved rod shown in Fig. 12. The nuts through which this rod turns, were bedded firmly in the wood, one of them being stripped of the thread and machined smoothly to act as a bearing. By turning the rod one way or the other, the position of the wire could be shifted very slowly in comparison with the turning of the rod.

When the small weights were lowered to the bottom of the shaft and replaced by the heavy plumb-bobs, these were brought to the required elevation by turning the reels with the cranks. They were held there by means of the pawls engaging the ratchet wheels as shown in Fig. 12. Any additional ad-

justment of the wires that might be required at the top after the heavy bobs had been attached, could still be easily made by turning the grooved rods.

Two transits were set up at the bottom of the shaft, on opposite sides of the plumb lines in the main tunnel excavation, and each transit was moved into exact line with the two suspended wires. The telescopes were then reversed to throw the line a greater distance from the bottom of the shaft, which was located "off center" from the main tunnel in such manner that 2 ft. of the shaft, along the longitudinal axis, would fall within the main tunnel excavation. The plumbed line was, therefore, an offset line which was corrected to true center by establishing a parallel tangent at the proper offset distance.

Determination of Length of Tunnel Axis

Chaining Legs of Traverse.—Measurements of the tunnel axis involved the determination of "distance" between two given points on the tunnel tangent, outside the portals, by means of an open traverse. A preliminary calculation was made from the two open traverses, which followed the general alignment of the operated line, a system of co-ordinates being used from which to compute "latitude" and "departure".

To determine "distance" in the most precise manner, an open traverse, consisting of four courses, was laid out on the ground and measured, beginning at a point on the tunnel axis produced east of the east portal, and ending at a point in the axis east of the west portal. (See Fig. 13.) One of the courses was laid through the old Cascade Tunnel, and produced eastward for some distance beyond the east portal of the old tunnel.

In laying out the courses through the timber, wide "cuttings" were made in order to secure unobstructed vision between the observation points at either end of each leg of the traverse. These "cuttings" extended for miles through fir, spruce, and hemlock, many of the trees being more than 3 ft. in diameter and 150 ft. in height. The actual measurement of the different "legs" of the traverse was done in the following manner: Substantial stakes were driven in the ground at proper intervals; a brass brad was placed in the top according to transit alignment, and the elevation of the top of the stakes determined by spirit leveling. The actual, or "slope" distance between tack points on the stakes was measured with steel tapes, a precise record being kept of the temperature and the tension on the tape at the time of measurement.

Two tapes were used on the survey; one, 500 ft. long, was used in measuring the old Cascade Tunnel, 2.63 miles in length, through which a part of one of the sides of the traverse was located; the other tape, 300 ft. long, was used entirely in the open.

These tapes were calibrated in Seattle by comparison with U. S. Government Standard, at Government Locks, Lake Washington Canal, at intervals of 100 ft. In addition to the calibration, experiments were conducted to test the standard formulas for lengthening due to tension and for shortening due to sag, with the tape in horizontal and inclined positions. These tests were made for the full length tape, and for 100-ft. fractional parts of it. From the data thus obtained, in order to facilitate the reduction of notes in the office,

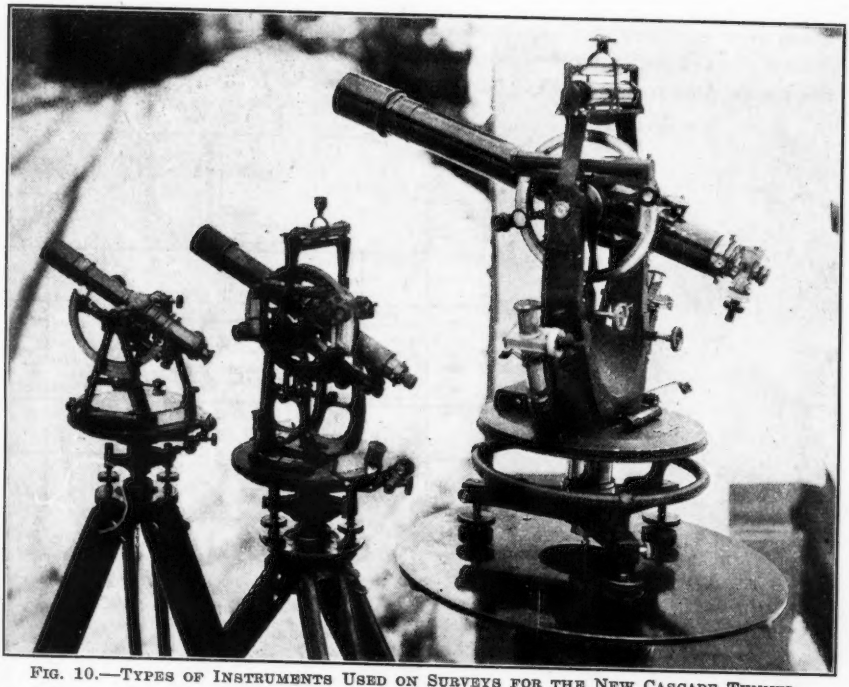

FIG. 10.—TYPES OF INSTRUMENTS USED ON SURVEYS FOR THE NEW CASCADE TUNNEL.

FIG. 11.—VIEW OF THEODOLITE MOUNTED AT ONE OF THE OBSERVATION STATIONS.

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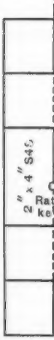

Fig

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graphs of the various corrections to be applied to the tape readings were made for each tape. These graphs were drawn on cross-section paper, to such a scale that all corrections could be easily read to the nearest thousandth of a foot.

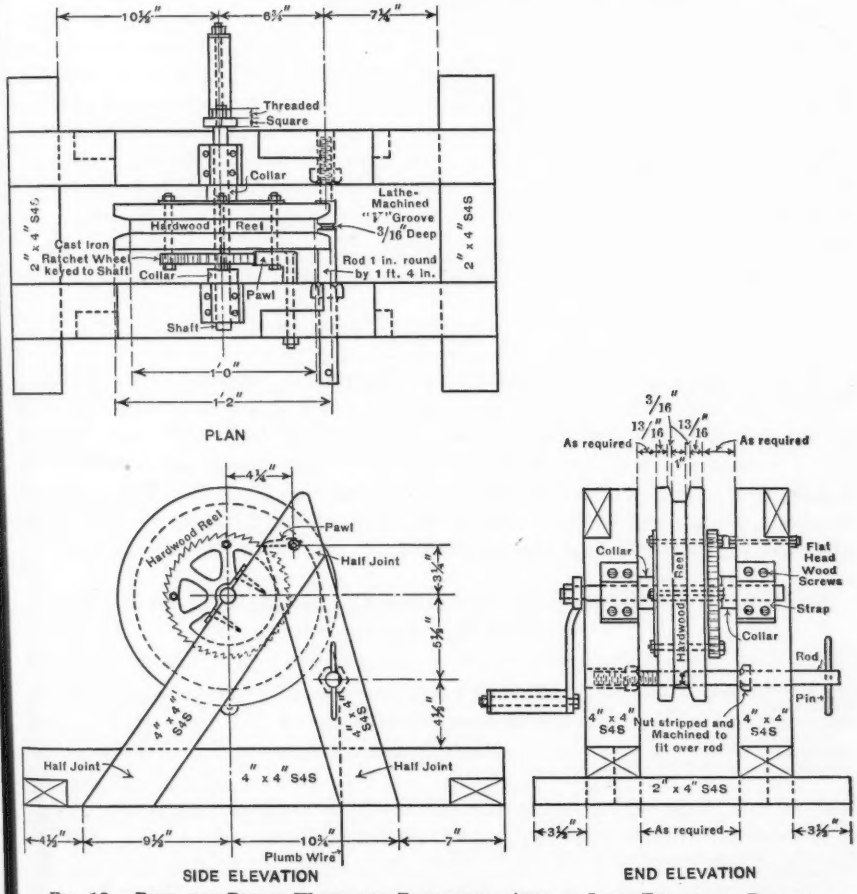


FIG. 12.—REEL FOR PLUMB WIRES FOR PROJECTING AXIS OF LONG TUNNEL AT BOTTOM OF MILL CREEK SHAFT.

The corrections applied were, as follows:

I.—A sag correction graph for the 300-ft. tape supported on level supports at the 0-ft. and 300-ft. marks, with the pull varying, in 10-lb. units, to 60 lb.

II.—A sag correction graph for the 500-ft. tape—conditions similar to those outlined for Correction I.

III.—A correction graph to apply to results taken from Corrections I and II, where tapes are supported by more than two level supports.

IV.—Graphs showing variation in elongation due to variation in pull.

V.—Graphs showing correction for temperature.

VI.—Graphs for corrections required in order to reduce measurements to a datum plane of 2 247 ft. above sea level, this datum being the elevation of the sub-grade at the west portal.

VII.—A graph for applying corrections for sag when the supports are not level, but on a uniform grade.

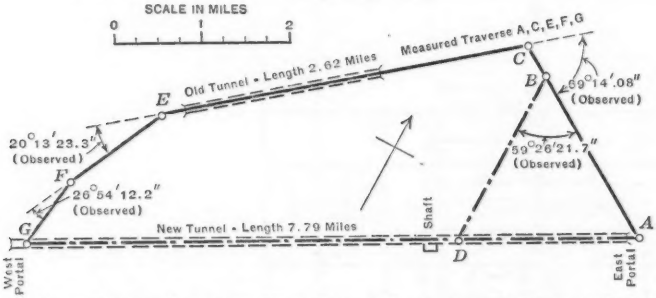


FIG. 13.—PRECISE TRAVERSE, NEW TUNNEL THROUGH THE CASCADE RANGE.

The following is a typical example of computation and applied corrections: The actual field notes for chained distance between Traverse Station No. 159 to No. 160 disclosed that the observed reading, with 30-lb. pull, was 299.900 ft. The tape was supported at five points, uniformly spaced, on a grade with a difference of elevation of outside supports equal to 37.40 ft. The temperature was 79° and the elevation of an end support was 2 904.06 ft. The computation was arranged as shown in Table 3.

TABLE 3.—COMPUTATION TO DETERMINE CORRECTED DISTANCE BETWEEN STATIONS NOS. 159 AND 160

Tape reading, in feet			299.900
Calibration correction	+ 0.033		
Sag correction, 97% (VII) times 0.22 in. (I) times 0.037 (III) times 3 (number of span)		− 0.025	
Tension correction (IV)	+ 0.053		
Temperature correction (V)	+ 0.067		
	+ 0.153	− 0.025 =	+ 0.128
Corrected slope distance, in feet			300.028
Corrected horizontal distance, $\sqrt{300.028^2 - 37.40^2}$			297.688
Correction for altitude			− 0.009
True horizontal distance at elevation datum, 2 247 ft., in feet			297.679

Observing Angles.—The precise transits, used in establishing the tunnel axis, were also used to observe the five angles of the precise traverse. The angles were observed in three sets of repetitions by two observers, each repetition consisting of three readings.

An exact record made of each angle reading was entered in the field book. The lengths of the courses were computed from these various field observations, curvature of the earth being disregarded, because the error involved in assuming the surface of the pentagon to be plane instead of spherical, was found to be less than 0.1 ft. for the longest side.

The short distances from Observation Stations *A* and *G*, Fig. 13, to the east and west portals, were measured by the same method used in measuring the sides of the precise traverse.

Triangulation Survey.—As a further check on the distance between portals and the relation of the Mill Creek shaft to either portal, it was decided to make a triangulation survey of the project. The work was carried out in the summer of 1926 by Henry B. Alvord, Assoc. M. Am. Soc. C. E.

The first course of the precise traverse near Berne, which had been measured by methods previously described, was used as the base line for the triangulation system. The results of the first and second measurements of the precise traverse, as compared with the triangulation survey, are shown in Table 4. In determining final measurement, one-third weight was given to the first measurement over the Great Northern traverse and two-thirds weight to the second measurement, the result of the triangulation being used for the purpose of comparison.

TABLE 4.—COMPARISON OF RESULTS

(H. B. Alvord's Results Proportioned to Conform with Observed Length of Base Line, *A C*, in Each Case)

Measurement	Observer	Length of Lines, in Feet		
		A D	*C E*	*A N*
First	Great Northern Engineers	.....	25 062 681	43 752.65
	H. B. Alvord	11 402.921	25 064.144	43 751.02
Second	Great Northern Engineers	11 400.358	25 061.449	43 750.22
	H. B. Alvord	11 401.451	25 060.913	43 745.38

Location of Mill Creek Shaft.—The 8 by 24-ft. shaft was "lined in" on the tunnel axis from two intermediate theodolite stations on the east and west slopes of Mill Creek Valley, shown in Fig. 9. The plumbing of the shaft has already been mentioned.

To determine the distance from the east portal to the shaft, the triangle, *A-B-D* (see Fig. 13), was laid out on the ground, the station, *B*, being a point on the base line, *A-C*. Station *D* was a point accurately set on the tunnel axis. The solution of the triangle, *A-B-D*, gave the length of the side, *A-D*, to which was added the measured distance from Station *D* to the mouth of the shaft.

Determination of Grade.—The west slope of the Cascades is everywhere much steeper than the east slope, the rise to the summit of the range being made in a shorter distance. In order to overcome this difference in natural ground slopes, the summit of the tunnel grade was placed at the east portal, which provided a longer distance on the west slope in which to make the climb to the summit, than would have been possible with the summit at some intermediate point between the portals. By following this plan, the same ruling grade was established over the engine district for eastbound and westbound movements.

A uniform grade of 1.565% was laid through the tunnel, descending from east to west. A long vertical curve of 0.10 variation per station connected the tunnel grade tangent with the 1.665% grade, ascending from east to west, on the new approach tracks to Berne.

When the elevation of the top of the shaft had been determined in its proper relation to the east and west portals, the sub-grade of the main tunnel was set 622 ft. below the top of the shaft. A steel tape was used to carry the measurement down the vertical side walls of the shaft.

Table 5 shows the comparison between the old and new line, between Berne and Scenic.

TABLE 5.—Comparisons Between the Old and New Lines Between Berne and Scenic

Items	Old line	New line	Favorable to new line
Length, in miles	17.67	9.99	7.68
Summit elevation, in feet (sub-grade)	3 382	2 881	501
Total rise westward, in feet	546	45	501
Total fall westward, in feet	1 325	824	501
Snowsheds, total length, in miles	6.04	0.00	6.04
Bridges, total length, in miles,	0.23	0.04	0.19
Tunnels, total length, in miles	3.66	7.79	(4.13)*
Maximum curve	10°00′	6°00′	4°00′
Total curvature	2 128°	187°	1 941°
Maximum grade, percentage	2.2%	2.2%	.....
Tunnel grade, percentage	1.695%	1.565%	0.13%

*Unfavorable to new line.

General Plan of Attack

The final plan for the construction of the Simplon Tunnel in the Swiss Alps was devised by the Jura-Simplon Railway Company in 1893, and was submitted, by an arrangement with the Swiss and Italian Governments, to a commission of experts, made up of Guiseppe Columbo, an engineer of Milan, Italy; Francis Fox, of London, England; and C. J. Wagner, of Vienna, Austria, State Inspector of Railways, formerly one of the engineers of the Arlberg Tunnel.

The plan provided for the driving of a secondary gallery, or pioneer tunnel, parallel to the main tunnel and 55.75 ft. from it. This pioneer tunnel was to be used to drain off underground water and to convey fresh air and cold water to the working faces. The plan had special merit in that it made it possible to enlarge the auxiliary gallery to a full-sized tunnel for double-track operation, whenever traffic conditions warranted such expenditure.

In formulating the plan for driving the New Cascade Tunnel, considerable study was given to the method of construction used on the Simplon Tunnel, and, as a result, it was decided to drive a pioneer tunnel between Mill Creek and the west portal.

The Mill Creek shaft divided the tunnel into two segments, the easterly portion being 2.41 miles in length, and the westerly portion, 5.38 miles. A study of time schedules showed that the section of tunnel between the east

portal and Mill Creek could be completed within three years by following the center-heading method, without the aid of a pioneer tunnel, but that the section of tunnel between Mill Creek and the west portal would require the use of an auxiliary haulage-way.

Accordingly, a pioneer tunnel was laid out between the shaft and the west portal, 66 ft. to the south of, and parallel with, the center line of the main tunnel, corresponding to the 55.75-ft. offset distance used in the Simplon Tunnel. (See Fig. 14.) It was considered that this interval provided sufficient distance away from the main tunnel to avoid disturbance to the rock formation without being too far away to complicate the construction of cross-cuts and the movement of spoil trains and construction supplies, back and forth between the pioneer and the main tunnel. In the event of future double track, the pioneer tunnel could be enlarged for second track, as was done in the Simplon Tunnel, or it could be used as a haulage-way from which to attack the new tunnel at intermediate points, as might be required. Likewise, the pioneer tunnel made permanent provision for taking care of the drainage from the main tunnel.

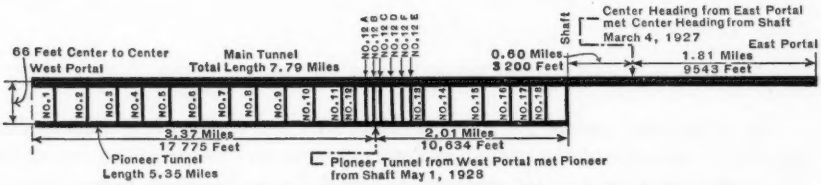


FIG. 14.—PLAN OF MAIN TUNNEL, PIONEER TUNNEL, AND CROSS-CUTS.

CONTROLLING FEATURES OF PROJECT

The controlling features of the project, which made it necessary to attack the work simultaneously at several intermediate points between portals, were as follows:

(1) Length of tunnel, 7.79 miles.
(2) Size of tunnel, single-track (see Fig. 15).
(3) Grade, 1.565%, descending east to west.
(4) Tunnel to be lined with concrete throughout.
(5) Possibility of encountering heavy volume of water.
(6) Time for completion, 3 years.

In formulating a plan of attack, every consideration had to be given the water problem, with the possibility of encountering water in such quantities that drifting operations, down grade, might be brought to a standstill.

The tunnel grade descended from east to west, and at the time of starting construction it was not known how successful the tunneling operations would be, if driven down grade. Judging from the rainfall in the winter months throughout the Cascade Range, with its numerous mountain streams and springs, it was reasonable to expect water-bearing rock formation in all primary penetration. These uncertain factors, together with the speed requirements of the time schedule, were responsible for the decision reached to sink the Mill Creek shaft and drive a pioneer tunnel west of the shaft. The shaft

opened up several new points of attack, and in case water in any considerable quantity was met between the east portal and the shaft, drifting operations could be continued up grade from the bottom of the shaft, the water encountered being drained by gravity into large sumps, and then lifted to the top of the shaft by electrically driven force pumps. Similarly, the pioneer tunnel, in advancing eastward (up grade) from the west portal, made it possible to drive the several cross-cuts to the main tunnel up grade, so that water from the enlargement operations would have gravity flow to the west portal.

Tye River Incline.—The surface indications at the west portal were of such a character that it was practically certain that soft ground would be encountered for a distance of about 1 000 ft. To avoid the delay which would result from excavating through this heavy ground, it was decided to drive a 30° incline adit, 251 ft. long, from the Tye River Valley to the pioneer tunnel, intersecting the pioneer center line and grade at a point 2 279 ft. from the west portal. Subsequent developments justified this move, as the material encountered in the pioneer tunnel, west of the adit, was of such a character that the advanced pioneer drifting crews, served from the Tye River incline, had penetrated to a point 5 703 ft. from the west portal by the time the west portal crews broke through into the advanced drift.

Underground Instrumental Control

To obtain the location of the tunnel portals for starting the underground operations, the axis of the tunnel, as determined by the line over the mountains, was transferred to a point near each portal by a direction theodolite (see Fig. 9).

In the case of the east portal, the line was thrown down from the theodolite set up on the mountain east of the east portal (near Berne), using the monument east of Mill Creek as a foresight. A transit was then set up over the "point" near the east portal, and the monument east of Berne was used as a back-sight to produce the line into the tunnel. At the west portal, corresponding operations were carried out and, in addition, a base line was established to control the pioneer tunnel. An offset line, 66 ft. parallel to the main tunnel axis, was laid out on the south side of the tunnel tangent, and this base line was used to project the line into the pioneer tunnel. In the case of the underground workings near the Mill Creek shaft, the 19-ft. base line, plumbed down from the top of the shaft, was used to carry the line forward toward the east and west portals.

All the advanced workings in the main tunnel between the west portal and the Mill Creek shaft, were controlled from the pioneer tunnel, both as to line and levels. As soon as the underground operations had advanced any considerable distance, more precise methods of establishing line and grade were used. In following up the main enlargement from the east and the west portals, the large direction theodolite, mounted on a 6-in. iron pipe support, was used to give the final line. On account of the heavy smoke, long sights were, as a rule, impossible. Leveling was done with levels having 24-in. telescopes, 4-ft. Philadelphia rods being used, where required.

In driving the 10 by 10-ft. center heading between the east portal and the Mill Creek shaft, under the instrumental controls previously described, the drifting crews met on March 4, 1927, at a point 3 200 ft. east of the shaft, 9 543 ft. west of the east portal. The error of closure was 0.23 ft. for "line", 0.2 ft. for "grade", and 1.9 ft. for "distance".

In driving the 8 by 9-ft. pioneer tunnel eastward from the west portal, and westward from the shaft, the two crews came together on May 1, 1928, at a point 17 775 ft. from the west portal. The error of closure was 0.64 ft. for "line", 0.78 ft. for "grade", and 1.0 ft. for "distance". (See Fig. 14.) This junction of the two control lines in the pioneer tunnel, well in advance of enlargement operations in the main tunnel, provided a ready means of checking back over the offset lines, connecting the pioneer with the main tunnel, so that a perfect tangent was established in the main tunnel, before the concrete lining was placed.

General Standards of Construction

Tunnel Section.—The internal width of the tunnel, between the concrete side walls, was determined from Great Northern tunnel standards which provided for a 16 ft. 0 in. width on tangent track. The internal vertical clearance did not follow the Great Northern standard of 22 ft. from top of rail to center of roof arch.

The electrification of the tunnel had been decided upon in the preliminary studies and in designing the tunnel section, a three-centered roof arch was provided. An 11 000-volt contact system was specified for present installation but, in planning the electrification, it had been decided to provide space for insulation against a future 22 000-volt trolley line and, in order to avoid the necessity of increasing the height of the roof arch, to provide for the longer insulators which would be required for the higher voltage, special plans were made for attaching the overhead trolley system to the roof of the tunnel, which reduced the distance between the trolley and the tunnel roof. Circular domes, 5 ft. 0 in. in diameter and 1 ft. 2 in. deep, were provided at the center of the concrete roof arch, at 75 ft. 0 in. intervals, from which the trolley insulators were suspended by means of a bronze insert, set accurately in place during the process of pouring the concrete lining.

The use of these domes and the three-centered roof arch was economical in that for a given height of trolley wire, as shown by Fig. 15, it resulted in reduction in quantity of rock excavation and diminished height of concrete side walls throughout the length of the tunnel.

The external horizontal and vertical axes of the tunnel section are variable according to the nature of the rock and the amount of excavation necessary to provide proper thickness of concrete lining. (See Fig. 15.) As a general rule, the following vertical and horizontal dimensions were used in the process of enlarging to full tunnel section:

Hard rock, requiring no timbering, 18 ft. 0 in. width by 25 ft. 0 in. above tunnel sub-grade.

Soft rock, requiring no timbering, 20 ft. 0 in. width by 25 ft. 6 in. above tunnel sub-grade.

Earth or rock, requiring timber lining, 22 ft. 0 in. width by 27 ft. 6 in. above tunnel sub-grade.

In placing the 12 by 12-in. timber lining, the timber sets were spaced, longitudinally, according to the character of the earth or rock.

Track.—Track is laid with Great Northern section, 110-lb. rail, and 7½ by 10-in. steel tie-plates, weighing 13.05 lb. each; 3 200 creosoted ties, 7 by 9 in. by 8 ft. long, are used per mile of track; 21 in. of washed gravel ballast was placed under the ties after the tunnel floor was built and leveled off to sub-grade with broken rock.

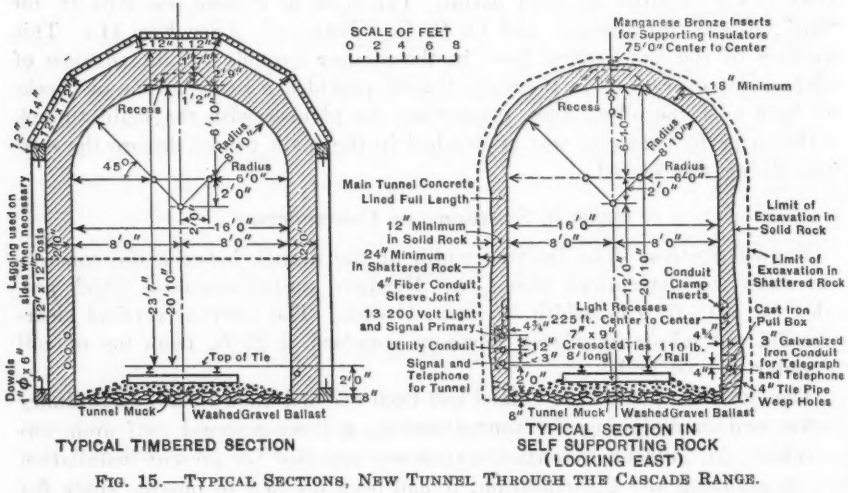

SCALE OF FEET
0 2 4 6 8
Recess
Radius 8'10"
Radius 6'0"
45°
2'0"
16'0"
8'0"
Lagging used on sides when necessary
Top of Tie
Tunnel Muck
Washed Gravel Ballast
TYPICAL TIMBERED SECTION
Manganese Bronze Inserts for Supporting Insulators 75'0" Center to Center
18" Minimum
Main Tunnel Concrete Lined Full Length
12" Minimum In Solid Rock
24" Minimum In Shattered Rock
4" Fiber Conduit Sleeve Joint
13 200 Volt Light and Signal Primary
Utility Conduit
Signal and Telephone for Tunnel
Limit of Excavation in Solid Rock
Limit of Excavation in Shattered Rock
Conduit Clamp Inserts
Light Recesses 225 ft. Center to Center
Cast Iron Pull Box
3" Galvanized Iron Conduit for Telegraph and Telephone
4" Tile Pipe Weep Holes
12" Creosoted Ties 110 lb. Rail
8" Tunnel Muck
Washed Gravel Ballast
TYPICAL SECTION IN SELF SUPPORTING ROCK (LOOKING EAST)

FIG. 15.—TYPICAL SECTIONS, NEW TUNNEL THROUGH THE CASCADE RANGE.

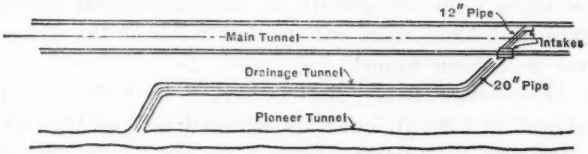
Drainage System.—The quantity of water encountered in underground operations differed greatly throughout the tunnel, and the permanent drainage system was designed to meet these varying conditions so as to provide ample capacity without unnecessary expense.

The east half of the tunnel was driven through comparatively dry formation, so that from the east portal westward for 3.85 miles, water, entering the tunnel from 4-in. weep-holes through the concrete lining along both sides of track, was carried in side ditches. In this distance, there was not a sufficient quantity of water to cause any trouble. At a point 3.85 miles from the east portal the accumulation of water was turned into the pioneer tunnel by means of a 12-in. pipe, embedded in the south wall of the concrete tunnel lining.

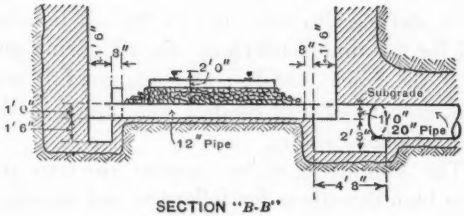
From this easterly cross-drain westerly, for a distance of 2.38 miles, the tunnel passed through a heavy, water-bearing formation, which required that cross-drains to the pioneer tunnel be constructed about every 1 500 ft. The pioneer tunnel, therefore, became the permanent artery of the drainage system, into which all water entering the main tunnel between the east portal and the most westerly cross-drain, a distance of 6.23 miles, was diverted. No cross-drains from the main tunnel to the pioneer tunnel were found necessary, from a point 1.56 miles east of the west portal to the west portal, because no heavy, water-bearing formation was encountered in this westerly section.

The bottom of the Mill Creek shaft was filled with concrete to a point 10 ft. above the top of the concrete tunnel arch. A 6-in. pipe was placed in

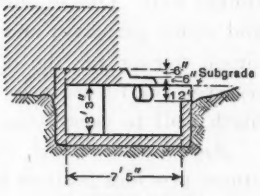
the concrete, discharging into the south side ditch of the main tunnel. The Mill Creek shaft was then completely back-filled with the broken rock, removed in the process of sinking the shaft.



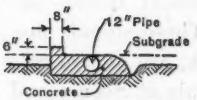
GENERAL PLAN



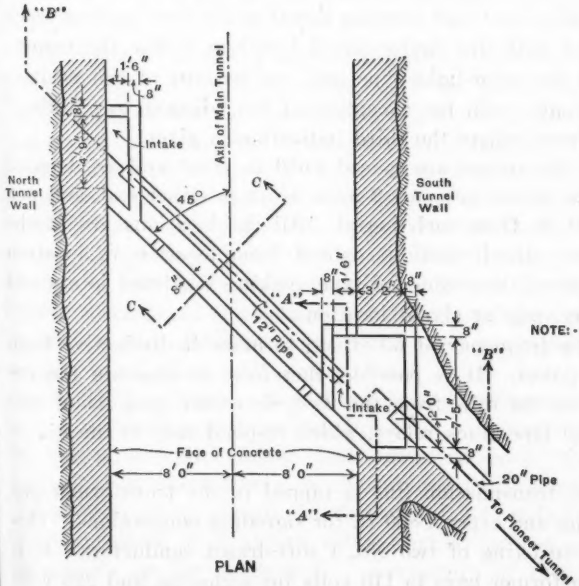
SECTION "*B-B*"



SECTION "*A-A*"



SECTION "*C-C*"



PLAN

FIG. 16.—PLAN SHOWING METHOD OF DIVERTING WATER FROM SIDE DITCHES OF MAIN TUNNEL TO PIONEER TUNNEL.

Fig. 16 shows the typical plan used to divert water from the main to the pioneer tunnel. The size of the cross-drain pipe was dependent upon the flow of water, 20-in. pipe being used in most cases. Thirteen permanent cross-

drains were built and these continue in service. At present, there are about 5 000 gal. per min. flowing out of the pioneer tunnel at the west portal, and steps are now being taken to maintain the pioneer tunnel, west of the first cross-drain (3.85 miles from east portal), by placing concrete lining in such sections as require it. Thus, there will be 3.94 miles of the pioneer tunnel in permanent use as a water tunnel.

Refuge and Transformer Bays.—Refuge bays, 8 by 8 ft., were placed in the south tunnel wall at 1 200-ft. intervals, within 1 mile of the east and the west tunnel portals and at 2 400-ft. intervals in the middle section of the tunnel. Transformer bays were placed in the north tunnel wall at 2 400-ft. intervals. (See Fig. 13.)

Three lines of 4-in. sleeve-joint fiber conduit were placed in the north tunnel wall. One of these conduits was used to carry the 13 200-volt lighting and signal primary; one was used for the tunnel telephone circuit wiring and signal wiring for the operation of the track circuits; and one conduit was reserved for future use. One 3-in. galvanized-iron conduit was placed in the south wall to carry telegraph and telephone cables.

Automatic Signal System.—The automatic signal system through the tunnel provides positive blocking in both directions for following and opposing train movements. It consists of an eastward starting signal at the east passing-track switch at Scenic and a westward starting signal at the west passing-track switch at Berne, together with five double-signal locations within the tunnel.

These signals are of the color-light type and, on account of the positive block, are of two colors only, with the exception of the "distant to entering" signals at Scenic and Berne, where the third indication is given.

The locations within the tunnel are spaced 9 600 ft. apart and are located so that one pair is at the center, one pair 2 miles in from either end, and one pair approximately 1 400 ft. from each portal. All the locations are at the transformer bays. Track circuit sections extend from location to location 9 600 ft. in length and are of the center-fed type, making insulated joints and reactance bonds necessary only at signal locations.

Power is supplied at a frequency of 60 cycles, to make it distinctive from the 25-cycle propulsion power. It is possible, therefore, to separate the 60-cycle signaling current on the rails from the 25-cycle return propulsion current by use of centrifugal type track relays, which respond only to the higher frequency.

The 13 200-volt signal transmission line is tapped in the transformer bay every 2 400 ft. for lighting and every 4 800 ft. for signaling connections. This is a lead-covered cable consisting of two No. 4 soft-drawn conductors. It is stepped down in the transformer bays to 110 volts for signaling and 220 volts for lighting. The cable is entirely sectionalized at the center bay and at the 2-mile bays. The track reactor bonds, with a capacity of 500 amperes per rail, which make the track continuous for propulsion current, but permit it to be cut into insulated sections for signaling current, are located in the transformer bays at the signal locations. The control circuits are carried in rubber-

covered wire through a fiber conduit in the tunnel wall and alternating-current line relays are used throughout.

The light signals are bolted directly to the tunnel wall so that their lowest point is at the level of the top of the rail, without encroaching on the side clearance. They are 8-volt 10-watt lamps, lighting directly from the alternating current, with a second or emergency light arranged to cut into the circuit automatically if the bulb in the green unit should burn out. All lights are operated on the approach lighting system. The two-color units show red for "Stop" and green for "Proceed", while the three-color units show red for "Stop", yellow for "Caution", and green for "Proceed".

Telegraph and Telephone Circuits Through Tunnel.—The railway and commercial telegraph and telephone circuits were carried through cable in a 3-in. galvanized-iron conduit in the south tunnel wall, below the floor level of the refuge bays. The iron conduit was used in order to get the benefit of its magnetic shielding effect, the usual lead sheath of the cable having no such magnetic qualities, although, of course, it does possess electrostatic shielding properties. The conduit run was broken every 600 ft. by cast-iron pull-boxes of special design, the cable being pulled in in 600-ft. lengths, and spliced at each box. The electrical continuity of the conduit was preserved past the box joints by spot-welded bond wires carried around each box. The entire conduit system was grounded, to provide effectual drainage of stray high-tension current.

The cable used was of American Railway Association, Telegraph and Telephone Section, Type H, 14 pairs, with the added dielectric strength of the double-paper insulation, providing for 2 000-volts breakdown between conductors, and 3 500 volts between conductor and sheath. All conductors were both paired and quadded, and all are 13 B. & S. gauge. Three quads, or six pairs of conductors, were equipped with loading coils, for telephone service. Load coils are located every 6 000 ft., or at every tenth pull-box, where an additional niche was cast in the tunnel lining of sufficient size to accommodate the coil on the top of the pull-box. As the load coils are enclosed in iron casings, no further mechanical or electrical protection was required for them. The loading coils reduce the impedance of the cable conductors to approximately that of open wire. The cable was extended at the portals sufficiently to run out even 6 000-ft. sections, and was terminated in fireproof tile and concrete cable houses, where it is protected against high voltages from outside the tunnel by two sets of standard carbon block protectors, with 14-ampere fuses, and with a set of gas protectors between the two sets of carbon protectors.

Communication locally in the tunnel is carried on through a separate rubber-insulated lead sheath cable, carried in the conduit with the signal secondaries in the north wall. Telephones of the weather-proof, iron-box, mine type are provided in each refuge bay, and the circuit is continued to the depot office near each portal, where necessary switching apparatus is provided to connect such tunnel circuit to the dispatcher's or other telephone circuits, as may be required. Extra conductors are also provided in this cable to take care of the dispatcher's telephone and to provide necessary test conductors, should the main cable be damaged beyond immediate repair. Each telephone

on the tunnel circuit is operated through an insulating transformer, to insure operation under adverse inductive conditions without danger to the user.

Tunnel Lighting.—Lighting recesses were placed at 112½-ft. intervals, "staggered", throughout the length of the tunnel. A plan was developed to utilize the 3/0 conductor No. 2 cable, which was used to furnish power for tunnel construction, as a means of providing for flood lighting at any point throughout the tunnel. These cables are attached to the side walls of the tunnel in such a manner that any part of the track can be lighted for the benefit of track gangs.

Trolley.—The 11 000-volt contact system throughout the tunnel is of simple catenary construction, consisting of a 350 000-cir. mils, stranded, copper messenger from which is suspended, by flexible loop hangers, a single 4/0 hard-drawn copper contact wire. The messenger is supported at 75-ft. intervals by a three-unit suspension type insulator and the entire contact system is sectionalized with an air gap and anchored at spacings of approximately 1 mile. Suspension type insulators were used throughout, both for messenger suspension and for strain insulators at the air gaps, with a voltage rating of 60 000.

Line and Grade Improvements, East Slope of Cascades

Coincident with the construction of the new Cascade Tunnel, and the electrification of the main line, a comprehensive program of grade and curve reduction and elimination of snow-sheds was carried out on the east slope of the Cascade Range between Peshastin (a station 19 miles west of Wenatchee) and Berne, near the east portal of the new tunnel. (See Fig. 2.)

The Chumstick Line.—The important part of this line revision was the construction of the new Chumstick Line from Peshastin to Winton, 16.92 miles, at a cost of nearly $5 000 000. The building of the Chumstick Line, so-called from the name of the creek valley which it follows for 7 miles, had been under consideration for a number of years prior to the construction of the new Cascade Tunnel.

In 1921, surveys were made for various routes *via* the Chumstick Valley, to avoid the Tumwater Canyon of the Wenatchee River. Most of these investigations contemplated turning off the old line near the Town of Leavenworth. The line that was constructed was a more radical improvement, in that better standards were used, with maximum curvatures of 3°, and a maximum grade of 1.6 per cent. In order to adhere to these standards, it was necessary to construct a spur line, 8 860 ft. long, to Leavenworth, to serve that community with carload freight.

The old main line followed the water grade of the Wenatchee River from its mouth to a point about 2 miles east of Winton, as shown in Fig. 2. The fall of the Wenatchee River between Leavenworth and its confluence with the Columbia River, is only 23 ft. per mile, but above Leavenworth, as far as Chiwaukum, the river passed through the Tumwater Canyon in a succession of rapids, where the rate of fall, in certain sections, is 100 ft. per mile. The old line was benched in on the north bank through this canyon, which required the use of 2.2% grades. For some distance through the canyon, the old track was protected against snow and rock slides from the upper slope by timber

sheds, a large percentage of which required renewal in 1928 and 1929. Of these sheds, 8 401 lin. ft. were already in place, and to protect the line adequately through the Tumwater Canyon, it was considered necessary to build approximately 4 000 ft. of new sheds. The large snow-shed program confronting the Railroad Company in the near future was a deciding factor in favor of the new route, but there were other advantages to be gained by the construction of the Chumstick Line, which may be briefly listed as in Table 6.

TABLE 6.—Advantages of Chumstick Line Compared with Old Line, Peshastin to Winton (16.92 Miles)

Item	Old line	New line	Saving
Length	18.08 miles	16.92 miles	1.16 miles
Curvature	2 039°-52′	622°-08′	1 377°-44′
Maximum curve	9°-30′	3°-00′	6°-30′
Number of curves	77	24	53
Maximum grade	2.2%	1.6%	0.6%
Length of snow-sheds	8 401 ft.	....	8 401 ft.
Length of tunnels	.....	7 448 ft.	.....

General Route.—Chumstick Line.—The new Chumstick Line turns off from the old line at Peshastin, and follows comparatively level bench lands until the Chumstick Valley is reached. The line then turns up this valley for about 7 miles, with very good general alignment; it penetrates a ridge, separating the Upper Chumstick Valley from the Wenatchee River Valley, by a tunnel 2 601 ft. long, and crosses the Wenatchee River ½ mile from the west portal of the tunnel on a 360-ft. steel bridge, at a point about 5 miles above the old line crossing, near Chiwaukum. For the next 2 miles, the line is laid along the slopes above the Wenatchee River, and follows Dead Horse Canyon for 1 mile, to a tunnel, 4 059 ft. long, through the ridge to Winton, where it connects with the old grade.

The maximum grade on this line is 1.6%, against westbound traffic, and is compensated for curvature. The first section of the 1.6% grade is 3 miles long, and the second section, 3½ miles long, and, except for these two stretches, the maximum grade is 1.5%, compensated.

Minor Line Revisions.—In addition to the new Chumstick Line, numerous short sections of the old line between Winton and Berne, a distance of 14 miles, were improved in alignment and grade. From Winton westward to Nason Creek (3 miles), a radical line change was made at a cost of about $500 000, which crossed and recrossed the old line, eliminating considerable curvature and improving the grade. There was a false summit on the old line at Winton which was approached by 2.2% grades westbound and 1% grades eastbound. This summit was lowered, and the sag to the westward was raised so as to provide a 1% westbound and a 0.6% eastbound grade.

Several other small line changes, in which a number of sharp curves were eliminated or reduced, were made on the east slope of the Cascades, by the distribution of 254 109 cu. yd. of tunnel spoil in a distance of 7 miles.

PART III.—CONSTRUCTION PLANS AND METHODS

BY J. C. BAXTER,[3] M. AM. SOC. C. E.

As stated in Part I, the snow-shed problem on the west slope of the Cascade Range early brought under consideration, by the Chief Engineers of the Railway Company, routes avoiding the snow-slide area. All these routes embodied long tunnels through the Cascade Range and in the later examination of these various projects, studies were made by the contractor (A. Guthrie & Co., Inc.), of the probable time required for driving the tunnels.

These studies were made with the primary thought of rapid progress:

First.—To save interest during construction.

Second.—Because additional savings could be made during the last year and possibly during the last two years of the construction period by not spending any money on replacement of snow-sheds.

Before proceeding with a detailed time study of any of the various tunnel projects, an examination was made of the methods that had been used for driving railway and mine tunnels, and of the probable drainage requirements of a long tunnel through the Cascade Range. From this examination the following statements were developed:

(*a*) The highest rate of progress in a long railway tunnel, as represented by total length divided by total time, had been made in the Simplon Tunnel, completed in 1906, and driven from an outside working gallery, or pioneer drift.

(*b*) The next highest rate of progress had been made in the Rogers Pass or Connaught Tunnel on the Canadian Pacific Railway in British Columbia, which was completed in 1916 and was probably the first example in the Western Hemisphere of the use of the Simplon outside gallery, or pioneer drift.

(*c*) The lowest cost per cubic yard for heading excavation obtained by the Contractor had been made in a double-track tunnel through shale in the Rocky Mountains on the Great Northern Railway, where working galleries on each side of the tunnel at the wall-plate elevation had been driven its entire length before proceeding with the heading excavation, which was then carried out from cross-cuts connecting the two working galleries, or pioneer drifts.

(*d*) The advantages of an outside working gallery could be obtained, under certain conditions, by the use of a shaft, as exemplified by the Ozark Tunnel, in Illinois, on the Illinois Central Railway.

(*e*) Where, as in the case of the Simplon Tunnel, the pioneer drift was afterward enlarged to a second railway tunnel, a pronounced saving in cost accrued, in addition to the increases in speed, and it was possible that a long tunnel through the Cascade Range might eventually require double track, or at least the construction of a passing track at its mid-length.

From a study of mining methods it appeared that most low-cost coal mining operations were conducted from working galleries or drifts. A study of the probable drainage requirements indicated that:

[3] Contr. and Engr. (Carey, Baxter & Kennedy), New York, N. Y., formerly Vice-Pres., A. Guthrie & Co., Inc.

(1) Practically all long tunnels driven in the past had developed large flows of water.
(2) The rocks of the Cascade Range, on account of the heavy annual precipitation, might be expected to carry water in large quantities.
(3) An outside working gallery, or pioneer drift, would facilitate the handling of water during construction, and provide an auxiliary drainage gallery into which any permanent flows of water could be diverted.

In consideration of the foregoing it was decided to assume a pioneer drift, or vertical shaft, where either were applicable, in making studies of the probable time that would be required to drive any of the projected tunnels.

These studies indicated that the tunnel on the route finally selected could be driven in three years. To arrive at this time the following plan had been outlined:

(*a*) The line of the tunnel was crossed at a point about one-third the distance from the east end by the Valley of Mill Creek, the tunnel grade at this crossing being only 622 ft. below the floor of the valley. A shaft at this point would cut the tunnel into two portions: An eastern section, 2.41 miles long, and a western section, 5.38 miles long; thereby reducing the continuous length of tunnel to be attacked to the length of the western section; that is, 5.38 miles.

(*b*) The western section if attacked from both ends, (through the west portal and the Mill Creek shaft) could be completed in three years, using a pioneer drift and center heading and removing all the enlargement muck from one face if the ground was favorable to enlargement from a center heading. If the ground was not favorable to such an operation, then the section could be completed in three years by as many secondary enlargement operations served through the pioneer drift as were necessary.

(*c*) The eastern section was obviously one that could be completed in three years in many different ways. However, it was almost certain the tunnel would require concrete lining throughout. There was no suitable aggregate for concrete near the Mill Creek shaft, which was two miles from the operated line of the railroad. The plan on this section was to speed it up by driving a center heading from both the east portal and the Mill Creek shaft, enlarging this section entirely from the east portal while securing access to the center heading from the Mill Creek shaft instead of a pioneer drift. In this way it would be possible to complete the eastern section early enough so that aggregate for the concrete lining of that section and of the section driven west from the Mill Creek shaft, could be delivered by rail at the east portal, which was practically on the line of the existing railroad.

(*d*) The ground from *A* to *B*, Fig. 17, consisted of the remains of an old slide, and it was certain that the penetration of this distance would be very slow. Therefore, an inclined shaft was proposed at *B* in the Valley of the Tye River, in order that the pioneer drift might be advanced continuously during the period that would be required to drive through from the west portal to *B*.

A time-distance chart based on this general plan is shown in Fig. 18, on which has been superimposed the actual progress in the various operations. After decision to construct the tunnel had been reached, this general plan was adopted and the tunnel successfully completed. However, this success was due

to the flexibility of the general plan adopted, in that it permitted wide variations of procedure where necessary to meet changing ground and water conditions. Sudden and great changes in ground and water conditions have been

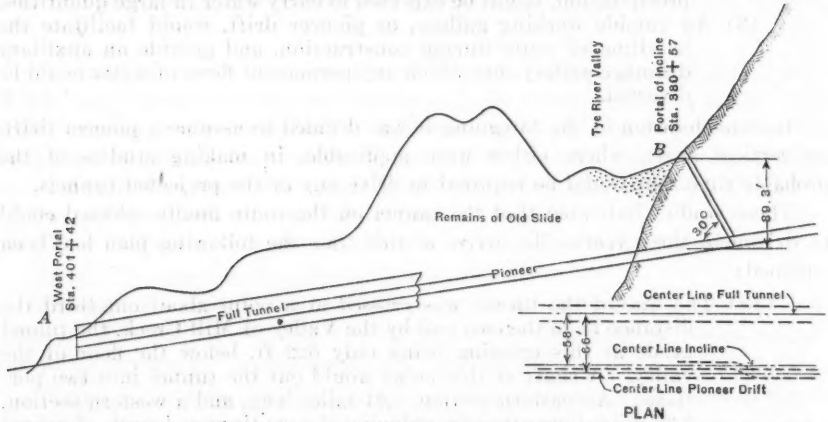


FIG. 17.—DIAGRAM SHOWING LOCATION OF TYE RIVER INCLINE AND REMNANTS OF OLD SLIDE.

typical of all long tunnel operations, and the Cascade Tunnel ran true to type, but, as mentioned, the basic plan of operation lent itself readily to meeting these contingencies.

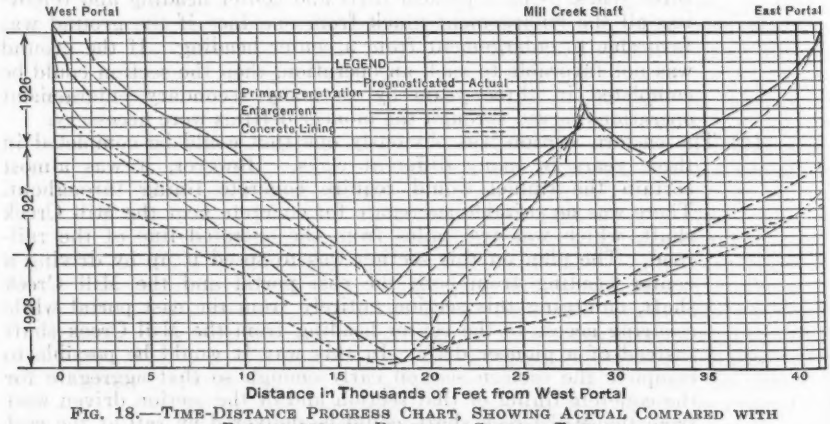


FIG. 18.—TIME-DISTANCE PROGRESS CHART, SHOWING ACTUAL COMPARED WITH PROGNOSTICATED PROGRESS, CASCADE TUNNEL.

To meet the various ground conditions encountered, seven general methods of attack were utilized, with combinations of some of them for short distances. These methods of attack and the conditions governing their use, were:

1.—Center heading, followed by enlargement (shown in Fig. 19). This was the basic method in the Cascade Tunnel plans. However, it only produces the best results where the ground is such that a full section of the tunnel will stand without timbering.

2.—Center heading, followed by top heading and bench (shown in Fig. 20). This method was used in ground where, after driving the center heading, it became apparent that the full tunnel section would not stand without timbering.

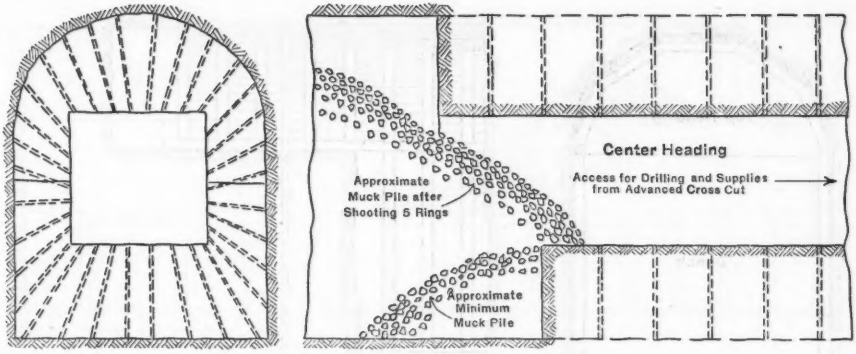

Center Heading
Access for Drilling and Supplies from Advanced Cross Cut
Approximate Muck Pile after Shooting 5 Rings
Approximate Minimum Muck Pile

FIG. 19.—DIAGRAM SHOWING METHOD OF ENLARGING TO FULL TUNNEL FROM CENTER HEADING.

3.—Top heading and bench (shown in Fig. 21). This method is the usual so-called "American Method" and was used in ground which, from indications in the pioneer drift, obviously would not stand without timbering.

4.—Wall-plate drifts, followed by top heading and bench (shown in Fig. 22). This is a modification of the method described under Method 3 and was used in ground too soft to stand for a full top heading.

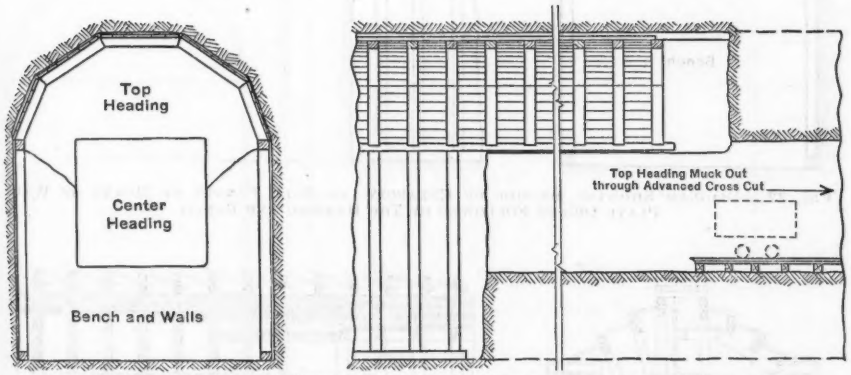


FIG. 20.—METHOD OF ENLARGING TO FULL TUNNEL FROM CENTER HEADING FOLLOWED BY TOP HEADING AND BENCH.

5.—Center top heading and wall-plate drifts simultaneously, followed by full top heading and bench (shown in Fig. 23). This is the so-called "Crown Bar Method" and was used for penetrating the old slide material at the west end of the Cascade Tunnel.

6.—Bottom heading followed by raises and top heading, the wings being removed by shovel (shown in Fig. 24). This is a method very similar to that under which the original No. 1 Tunnel of the Simplon was driven, and was made use of in the Cascade Tunnel in order to expedite completion by introducing hand-tunneling operations through several of the cross-cuts after the pioneer was holed through.

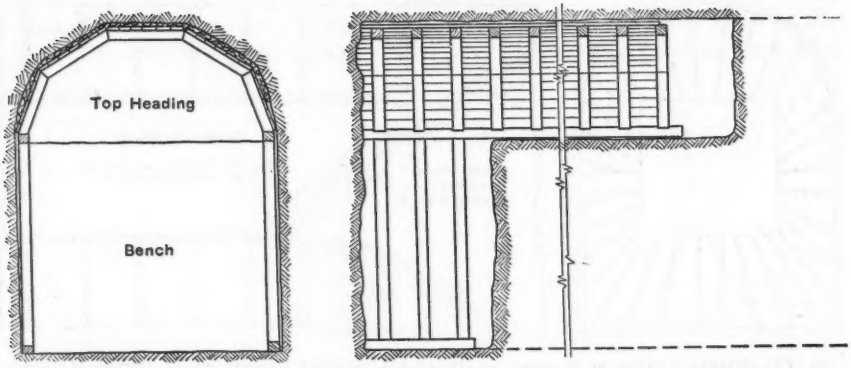


FIG. 21.—DIAGRAM SHOWING TOP HEADING AND BENCH METHOD OF DRIVING FULL TUNNEL.

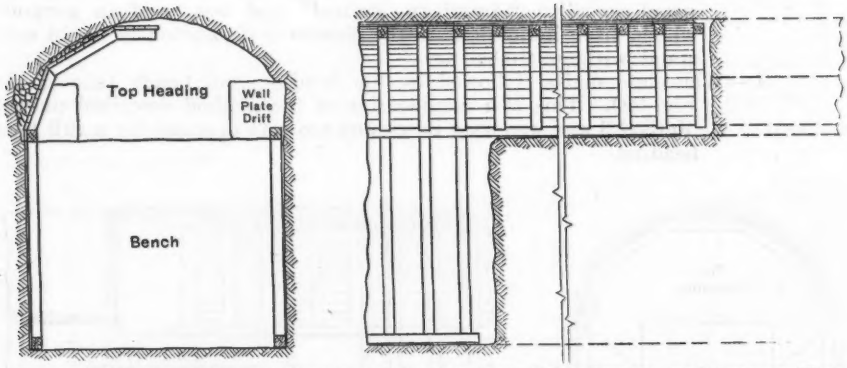


FIG. 22.—DIAGRAM SHOWING METHOD OF ENLARGING TO FULL TUNNEL BY MEANS OF WALL PLATE DRIFTS FOLLOWED BY TOP HEADING AND BENCH.

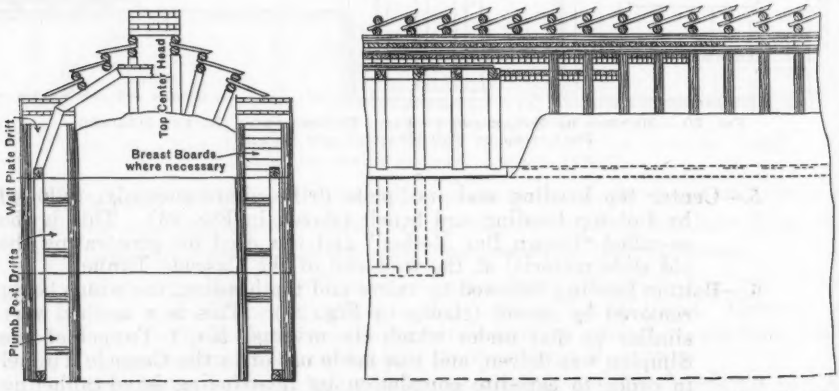


FIG. 23.—DIAGRAM SHOWING METHOD OF ENLARGING THE FULL TUNNEL FROM TOP CENTER HEADING AND WALL-PLATE DRIFTS.

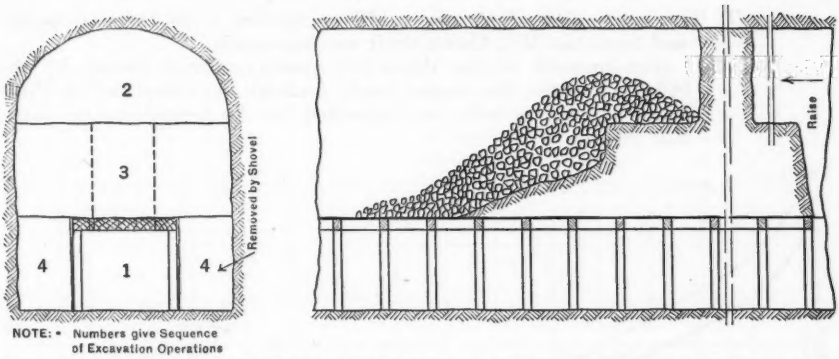


FIG. 24.—DIAGRAM SHOWING METHOD OF ENLARGING TO FULL TUNNEL FROM BOTTOM CENTER HEADING BY TOP HEADING AND RAISES.

7.—Full tunnel face (shown in Fig. 25). This is a method seldom used in tunneling, but was adopted in the Cascade Tunnel in order to expedite completion by providing additional mechanical loading places, reached through cross-cuts from the pioneer drift in advance of the main enlargement operation. The procedure is to shoot 1*A* first and 1*B* immediately thereafter. The independent round is then drilled and shot as soon as ready.

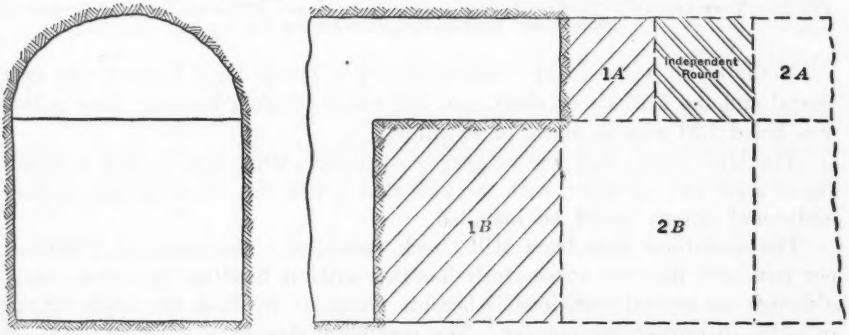


FIG. 25.—DIAGRAM SHOWING METHOD OF DRIVING FULL TUNNEL FACE.

DRAINAGE

When laying out the plan of operations the problem of water to be encountered was given careful consideration along the following lines:

(*A*) The tunnel was located through the Cascade Range in an area of high precipitation, cut by many sturdy mountain streams and containing many springs, several of which in the near vicinity were warm. It was, therefore, almost certain that water in large but unknown quantities would be encountered.

(*B*) The tunnel grade was all in one direction, down hill from the east portal.

(*C*) Water in any quantity would slow up the operation, but in large quantities (from 3 000 to 5 000 gal. per min.) would probably cause the abandonment of an operation being carried out down hill.

(*D*) The continuity of the down-hill operations from the east portal and from the Mill Creek shaft was uncertain.

(*E*) If abandonment of the down-hill operations was forced by the inflow of water, the tunnel could probably be completed in three years and six months (as indicated by the time-distance chart, Fig. 26).

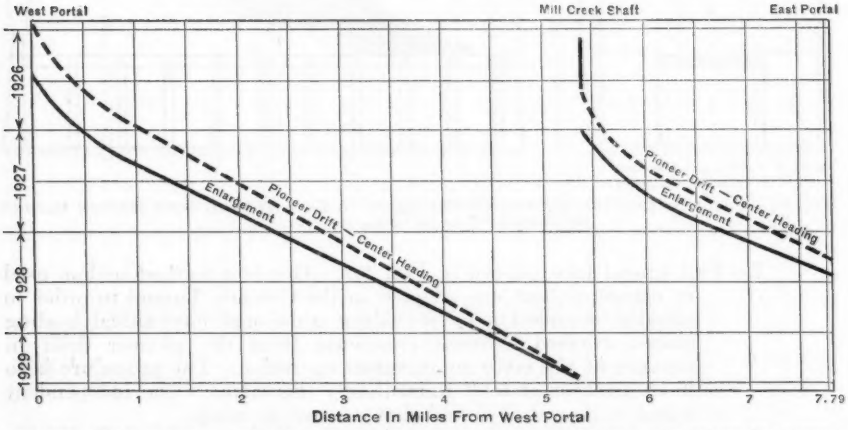


FIG. 26.—TIME-DISTANCE CHART SHOWING PROBABLE PROGRESS BASED ON UPGRADE OPERATION FROM WEST PORTAL AND MILL CREEK SHAFT.

Actually no water of any consequence was encountered between the east portal and the Mill Creek shaft, and the center heading between these points was holed 2.93 months ahead of schedule.

The Mill Creek shaft was originally equipped with pumps having a capacity of 2 200 gal. per min., with the idea that before this capacity was reached additional pumps would be installed.

The operations west from Mill Creek developed a maximum of 2 800 gal. per min. and this was successfully handled without flooding the entire work, although on several occasions it became necessary to flood the main tunnel in order to protect the pioneer. The maximum flow from a single point of discharge in the operations west of Mill Creek was 800 gal. per min. This flow, added to the other water then being handled, threatened to flood the work, but was shut off by a concrete bulkhead and developed against the bulkhead a pressure of 100 lb. per sq. in. The bulkhead was fitted with an outlet pipe and valve and the water behind this bulkhead was gradually drained to the pumps at a rate they could handle. A curve showing the total water discharge from the Mill Creek shaft is given in Fig. 27.

Fortunately, the worst water conditions were encountered in the operations being driven uphill from the west portal where the water could flow out by gravity and where a maximum discharge of 10 300 gal. per min. was developed. Practically all the water encountered was ground-water filling fissures in the rock. These fissures were of all sizes—from hair-cracks to 6 or 8 in. across. All these fissures acted in the same manner: A maximum flow when first encountered; a radical decrease in the flow in the next few days (sometimes

hours), with a gradual decrease over weeks and months, at times drying up entirely.

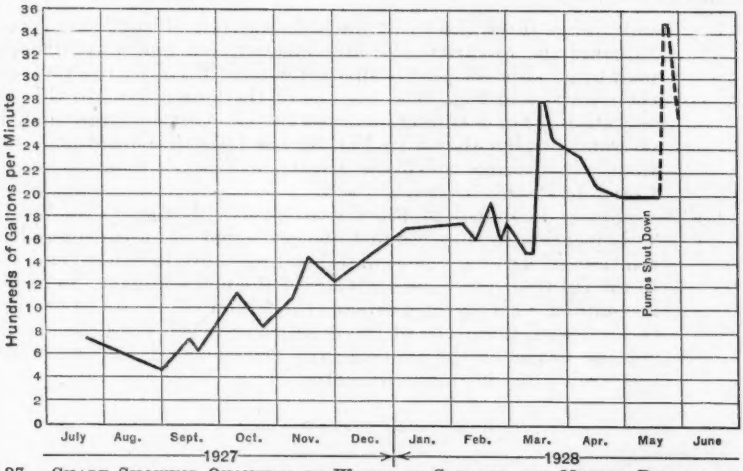


FIG. 27.—CHART SHOWING QUANTITY OF WATER IN GALLONS PER MINUTE PUMPED AT MILL CREEK SHAFT. BROKEN LINE INDICATES FLOW TO WEST PORTAL.

Very high pressures were occasionally encountered in drilling, but were never measured accurately; for instance, a 12-ft. drill steel, 1¼ in. in diameter, when released from the drill, was forced out of the hole by the water pressure, the steel passing back over the drill carriage and injuring a nipper who was about 15 ft. back from the face.

The maximum flow from a single point of discharge in the west portal operations was 5 000 gal. per min., but this dropped rapidly within 24 hours. A curve showing the total water discharge from the west portal operations is given in Fig. 28.

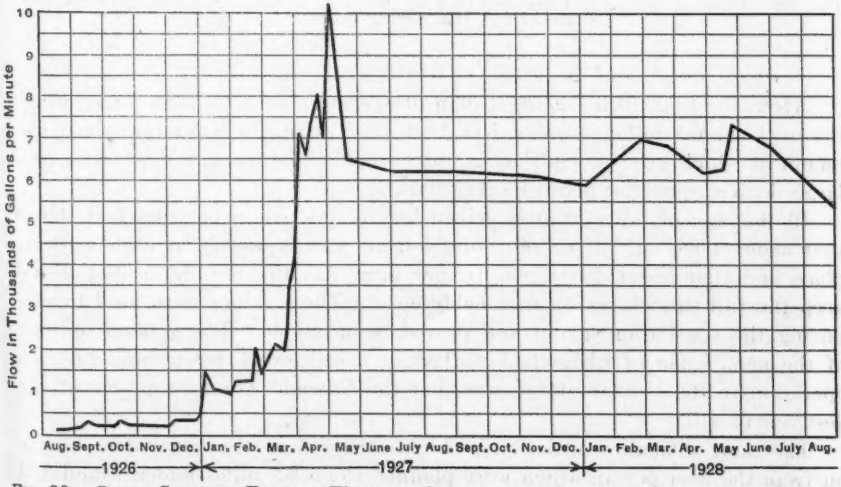


FIG. 28.—CHART SHOWING FLOW OF WATER IN GALLONS PER MINUTE AS MEASURED AT PORTAL OF SCENIC PIONEER TUNNEL.

The original plan of the pioneer tunnel is shown in Fig. 29. This was wrong in two respects:

1.—The drainage ditch shown at the side was not of sufficient capacity to carry the quantity of water encountered and was difficult to maintain. Fig. 30 shows the water conditions in the pioneer at their worst and Fig. 31 shows one of the larger flows in this same operation. For a tunnel expected to take care of large quantities of water, a plan shown in Fig. 32 has been developed and is now in use in driving the Mahr Drainage Tunnel, 6 miles long, for Cerro de Pasco, in Peru.

2.—The pioneer is shown in Fig. 29 as level with the center heading. This location is simple and to be preferred when considered with relation to driving the center heading, but does not lend itself to the future permanent drainage of the main tunnel. In order to accomplish this latter purpose, the grade of the pioneer should be level with the grade of the main tunnel. In the Cascade Tunnel the grade of the pioneer was dropped and 1.32 miles of it are level with the main tunnel grade.

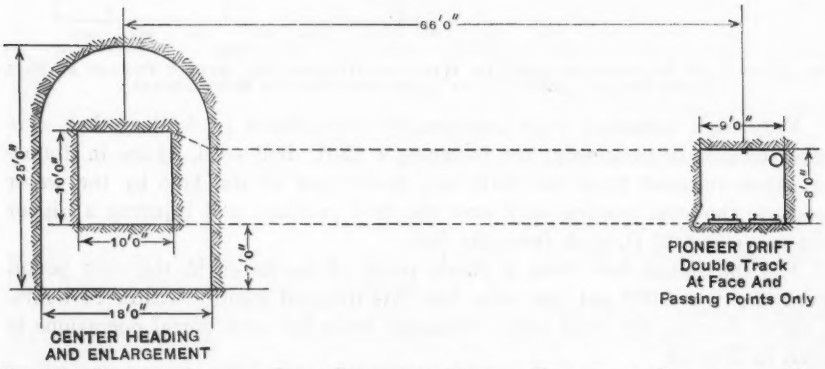


FIG. 29.—DIAGRAM SHOWING CROSS-SECTION OF PIONEER TUNNEL AND RELATION TO MAIN TUNNEL AS ORIGINALLY PLANNED.

VENTILATION

After deciding upon a general plan of operation for driving a long tunnel, the next subject to be considered is the tunnel plant, the most important elements of which are the ventilating units, which, in turn, are dependent to a large degree upon the plan of operations.

In the case of other tunnels driven before this one, experience had shown that about 4 000 cu. ft. per min. of fresh air was necessary at each working place and that about 20 000 cu. ft. per min. was the minimum that would keep the full tunnel section reasonably clear. These values were used in preparing the ventilation layout and proved to be ample. The general opinion of the men, some of whom had worked in nearly every recent underground operation in the United States, was that the Cascade Tunnel was one of the best ventilated.

The most difficult ventilation problem occurred in the operations carried on from the west portal, which were planned to go 3.5 miles underground and actually were carried forward 3.36 miles before the hole through.

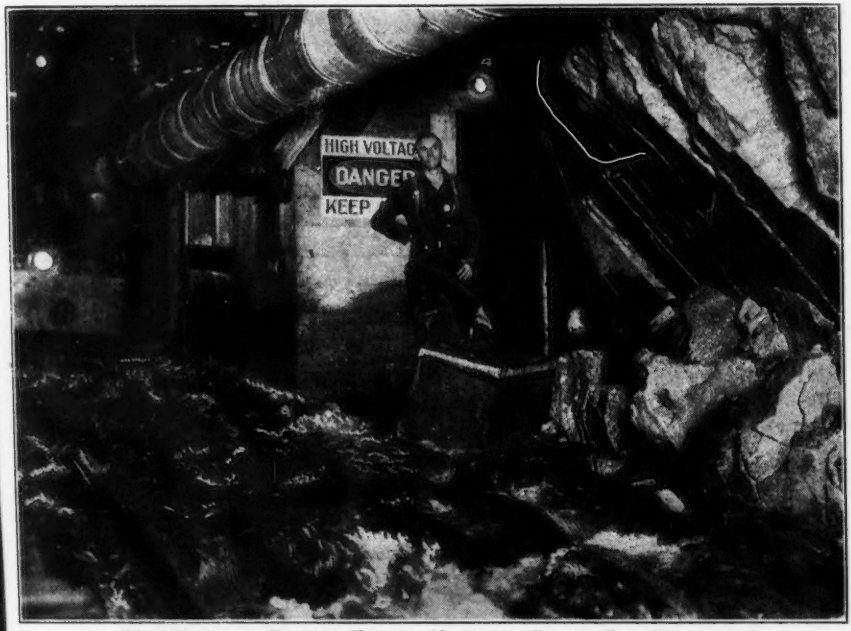
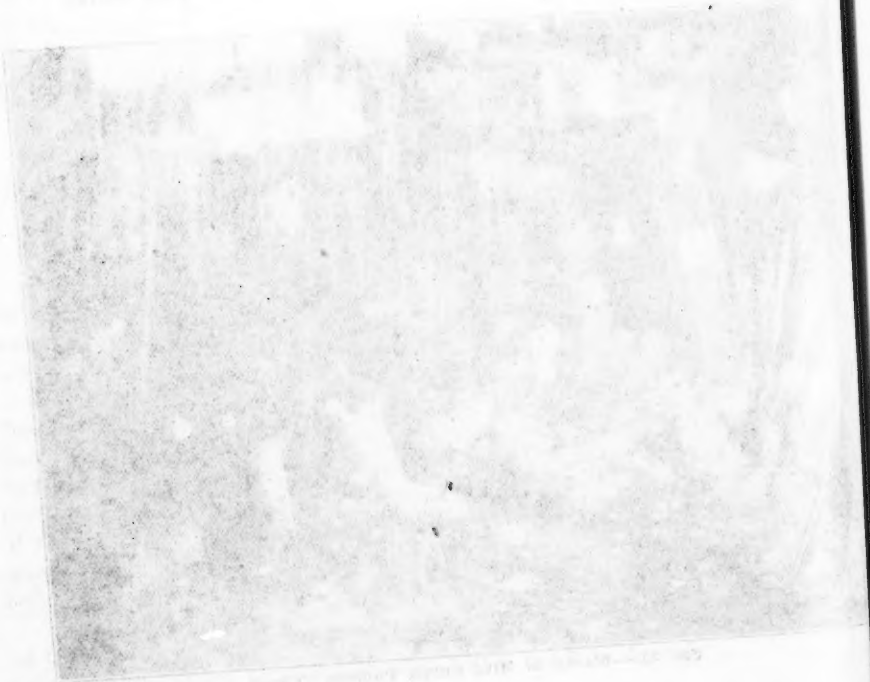


FIG. 30.—WATER IN PIONEER TUNNEL NEAR THE SCENIC END, WEST PORTAL.

FIG. 31.—WATER IN MILL CREEK PIONEER TUNNEL.



The plan adopted at the west portal was to bulkhead the pioneer and force air into it with low-pressure fans in volumes ranging from 20 000 to 30 000 cu. ft. per min., the air passing through the pioneer to some advanced cross-cut where a bulkhead was to be erected, diverting the air current through the cross-cut into the main tunnel and thence out to the portal. The cross-cuts between the portal and the advanced bulkhead in the pioneer were to be closed. This plan provided the primary ventilation.

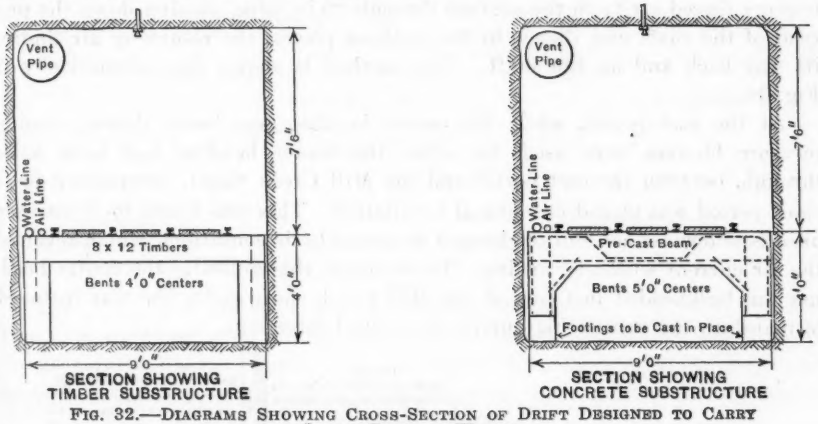


FIG. 32.—DIAGRAMS SHOWING CROSS-SECTION OF DRIFT DESIGNED TO CARRY LARGE FLOW OF WATER.

At an advanced bulkhead in the pioneer was to be installed a high-pressure fan, capable of delivering 12 000 cu. ft. per min. against a static head of 20 to 30 in. of water, taking air from the primary ventilation current and delivering it through a 20-in., 18-gauge, riveted and soldered ventilating pipe to the various advance heading operations, at the rate of about 4 000 cu. ft. per min. for each working place. Furthermore, at the advance bulkhead an equalizing fan was to be installed to pass the returning air through the bulkhead back into the primary ventilating current. This plan is shown diagrammatically in Fig. 33 and worked perfectly, with the following exceptions:

(*a*) The use of an advance bulkhead and the equalizing fan was found unnecessary. The primary ventilating current was simply allowed to find its way through the only cross-cut remaining open from the pioneer to the main tunnel and the high-pressure fan was set at this cross-cut, drawing fresh air from the primary

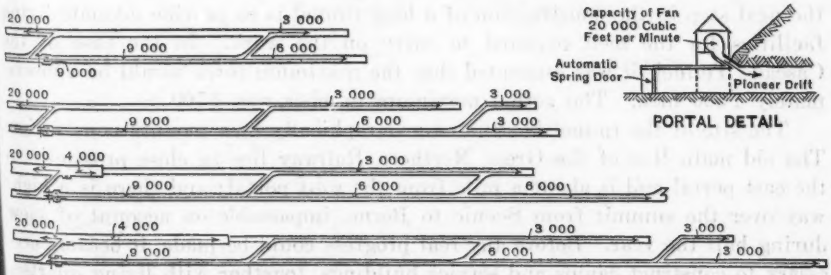


FIG. 33.—DIAGRAM SHOWING VENTILATION SCHEME USED IN WEST PORTAL OPERATIONS.

current at a point portal-ward from the cross-cut, the returning air finding its way back into the primary current.

(*b*) The high-pressure fans that were available, while delivering their rated capacities against the rated static heads, were too lightly constructed for the service and required heavy repairs. This is a detail, however, that can easily be corrected by the fan manufacturers in future installations of this character.

At the operations east and west from the Mill Creek shaft common pressure blowers forced air from the surface through 20-in. pipe, passing down the pipe road of the shaft and thence to the working places, the returning air finding its way back and up the shaft. This method is shown diagrammatically in Fig. 34.

At the east portal, while the center heading was being driven, similar pressure blowers were used, but after the center heading had been holed through, between the east portal and the Mill Creek Shaft, dependence for a short period was placed on natural ventilation. This was found to be entirely unsatisfactory on account of changes in atmospheric conditions, which reversed the air current without warning. To overcome this difficulty the center heading was bulkheaded just east of the Mill Creek shaft and a fan was installed to maintain the ventilating current in a fixed direction.

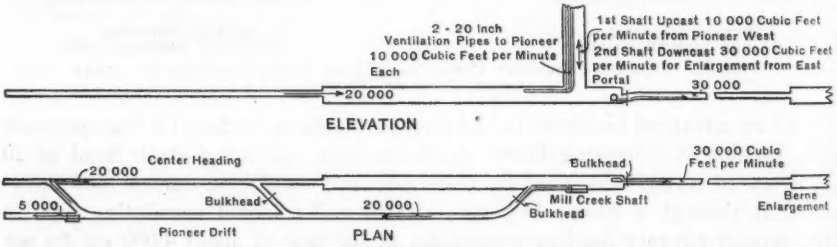


FIG. 34.—DIAGRAM SHOWING VENTILATION SCHEME FOR OPERATIONS AT MILL CREEK SHAFT AND BERNE ENLARGEMENT.

All ventilating pipes were 20-in., 18-gauge, galvanized, with riveted and soldered seams, the length being connected by a special packed joint, as shown in Fig. 35, while for changing the air current from blowing to suction, reversing gates, as shown in Fig. 36, were used.

CAMPS AND FACILITIES

After having decided upon a plan of attack and a method of ventilation, the next step in the construction of a long tunnel is to provide adquate living facilities for the men required to carry on the work. In the case of the Cascade Tunnel, it was estimated that the maximum force would be approximately 1 500 men. The actual maximum number was 1 700.

The site of the tunnel is situated geographically in a mountainous region. The old main line of the Great Northern Railway lies in close proximity to the east portal and is about a mile from the west portal; and there is a highway over the summit from Scenic to Berne, impassable on account of snow during half the year. Before any real progress could be made, it became necessary to construct camps and service buildings, together with living quarters

for married employees, at all points of attack. It was also necessary to construct a road, about 3 miles long, from Berne on the main line of the railroad to the Mill Creek shaft.

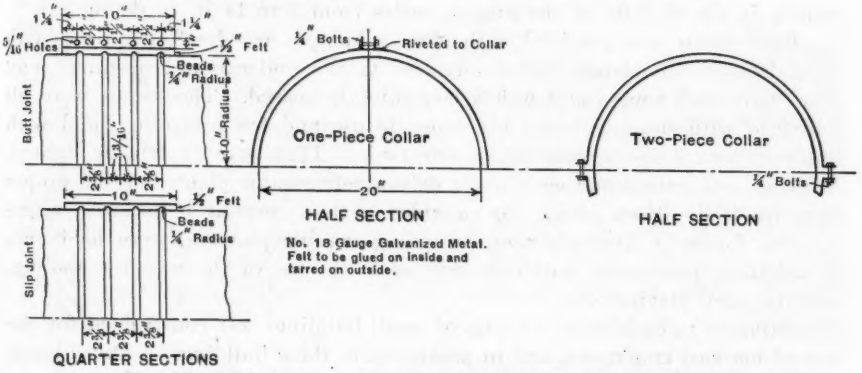


FIG. 35.—PIPE JOINTS ON VENTILATION SYSTEM, CASCADE TUNNEL.

The difficulty of constructing these camps and the road will be appreciated from the statement that timber standing, in some instances, 50 000 ft. b. m. to the acre had to be cleared and disposed of before the work could proceed. Fig. 37 is a view of a portion of the west portal camp and shows the stand of timber clearly. Fig. 38 shows the camp and family quarters, at Berne.

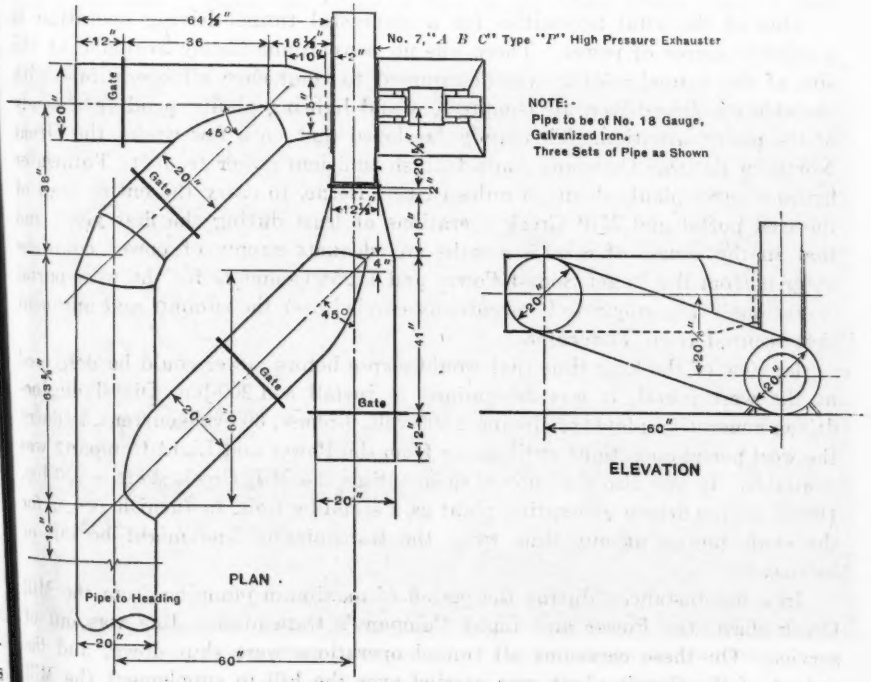


FIG. 36.—PIPE ASSEMBLY FOR AIR CONTROL, CASCADE TUNNEL.

The buildings in all the camps were erected 6 to 10 ft. above the ground and connected with sidewalks for two reasons: (1) To avoid the tremendous expense of grubbing the natural surface; and (2) to aid in the disposal of snow which, in the vicinity of the tunnel, varies from 2 to 14 ft. in depth.

Each camp was provided with change houses, or "drys", equipped with steel lockers and shower baths, adjacent to the underground openings, and also with wash-houses and toilets conveniently spaced. The camps were all provided with running water for domestic use and fire protection, and with sanitary sewers discharging into septic tanks. They were electrically lighted, and the cook camps had electrically driven refrigerator plants for the proper care of food. Each camp was provided with a recreation building where various forms of entertainment, including moving pictures, were held and, in addition, there were buildings devoted to the use of the men for reading, writing, card playing, etc.

Adjacent to each camp a group of small buildings was constructed for the use of married employees, and in proximity to these buildings, a school-house and school teachers' dwelling. Each camp was provided with first-aid stations and an emergency hospital, presided over by a resident physician. Fig. 39 shows the general type of camp construction; Fig. 40 the type of toilet and change houses; and Fig. 41 shows the type of quarters provided for married employees.

Power

One of the vital necessities for a successful tunnel-driving operation is a reliable source of power. There was no power immediately available at the site of the tunnel, and it was determined to commence all operations with portable gasoline-driven air compressors and lighting plants, pending a study of the power situation. This study developed that, in a few weeks, the Great Northern Railway Company could furnish sufficient power from its Tumwater hydro-electric plant, about 25 miles east of Berne, to carry the entire load of the east portal and Mill Creek operations at least during the first year, and that in the course of 8 or 9 months an adequate supply of power could be secured from the Puget Sound Power and Light Company for the west portal operations—this supply to be eventually carried over the summit and augment that secured from Tumwater.

In view of the long time that would elapse before power could be delivered at the west portal, it was determined to install a 1 200-h.p. Diesel engine-driven generating plant, producing 2 300-volt, 3-phase, 60-cycle current, to carry the west portal operations until power from the Power and Light Company was available. It was also determined to install at the Mill Creek shaft a 720-h.p. Diesel engine-driven generating plant as a stand-by unit, to furnish power for the shaft pumps at any time when the transmission line might be out of service.

In a few instances, during the period of maximum pumping from the Mill Creek shaft, the Power and Light Company's transmission line was out of service. On these occasions all tunnel operations were shut down, and the output of the Scenic plant, was carried over the hill to supplement the Mill

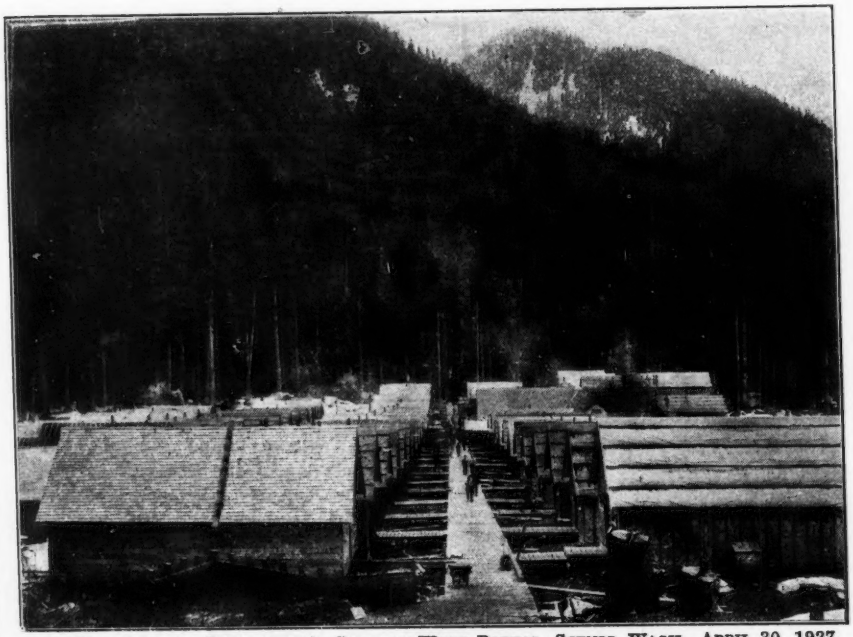


FIG. 37.—VIEW OF CONTRACTOR'S CAMP AT WEST PORTAL, SCENIC, WASH., APRIL 30, 1927.

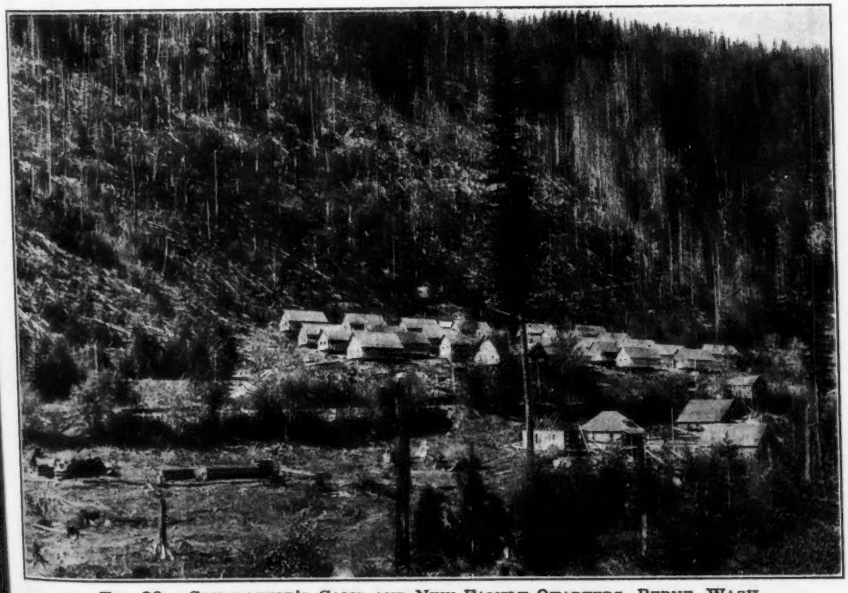


FIG. 38.—CONTRACTOR'S CAMP AND NEW FAMILY QUARTERS, BERNE, WASH.

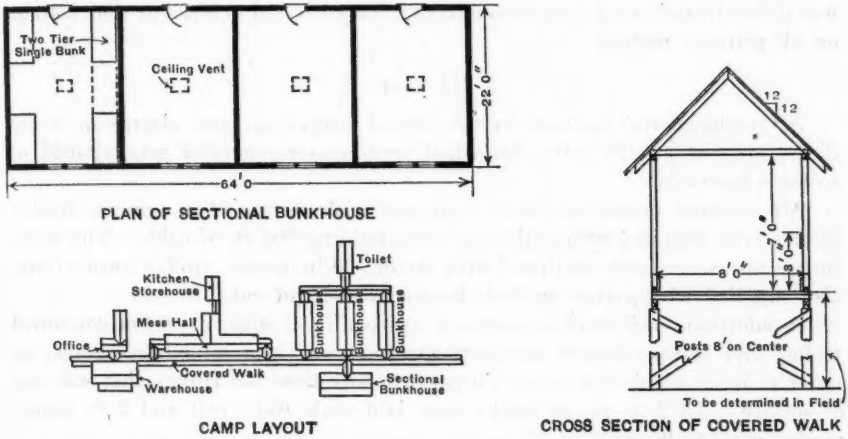


FIG. 39.—DIAGRAM SHOWING GENERAL TYPE OF CAMP CONSTRUCTION, CASCADE TUNNEL.

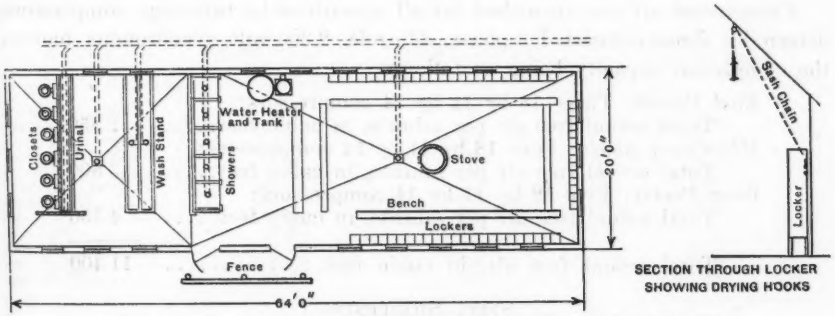


FIG. 40.—DIAGRAM SHOWING TYPE OF TOILET AND CHANGE HOUSE.

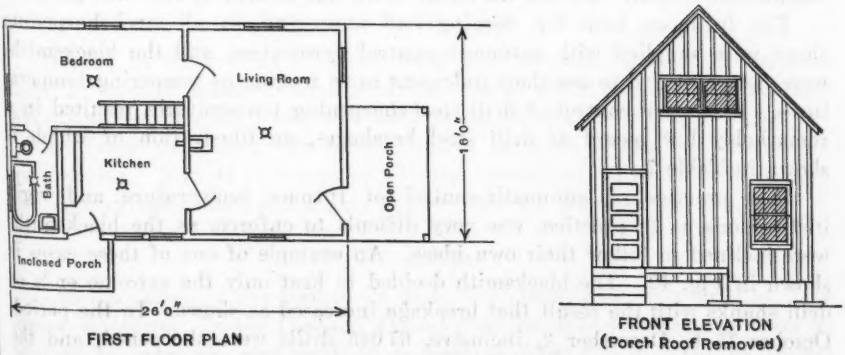


FIG. 41.—DIAGRAM SHOWING TYPE OF QUARTERS PROVIDED FOR MARRIED EMPLOYEES.

Creek plant and thus carry the pumps in the Mill Creek shaft. All power was either transformed to or produced at 2 300 volts and utilized at that voltage on all primary motors.

Haulage

All underground haulage in the tunnel operations was electrical, using direct current at 275 volts, furnished from motor-generator sets, spaced at suitable intervals.

All drifting operations were equipped with 6-ton, 24-in. gauge, trolley locomotives, supplied with gathering reels, holding 500 ft. of cable. The main tunnel operations were equipped with 20-ton, 36-in. gauge, trolley locomotives, also supplied with gathering reels holding 500 ft. of cable.

In addition, each working opening was equipped with one 6-ton combined trolley and storage-battery locomotive, which was kept on outside work, in order to be available for rescue purposes at any time the trolley line was out of service. All 2-ft. gauge tracks were laid with 40-lb. rail and 3-ft. gauge tracks, with 60-lb. rail.

Compressors

Compressed air was furnished for all operations by two-stage compressors, driven by direct-connected 3-phase, 60-cycle, 2 300-volt synchronous motors, the compressor capacity being as follows:

East Portal: Three 18 by 11 by 14 compressors: Total actual free air per minute, in cubic feet.....	2 850
Mill Creek Shaft: Four 18 by 11 by 14 compressors: Total actual free air per minute, in cubic feet.....	3 800
West Portal: Five 18 by 11 by 14 compressors: Total actual free air per minute, in cubic feet.....	4 750
Total actual free air, in cubic feet.....	11 400

Steel Sharpening

Steel sharpening shops were established at each camp and a general machine and repair shop for the entire work was located at the west portal.

The furnaces, both for forging and tempering, in all steel-sharpening shops were supplied with automotic-control pyrometers, and the blacksmiths were not permitted to use their judgment as to forging or tempering temperatures. This close control of drill steel-sharpening temperatures resulted in a remarkably low record of drill steel breakages, an illustration of which is shown in Table 7.

This practice of automatic-control of furnace temperature and rigid instructions as to practice, was very difficult to enforce, as the blacksmiths were inclined to follow their own ideas. An example of one of these cases is shown in Fig. 42. The blacksmith decided to heat only the extreme ends of drill shanks with the result that breakage increased as shown. In the period, October 15 to December 3, inclusive, 67 045 drills were sharpened, and the number broken during the same time was 1 672. The total average breakage was 2.49%, as compared with the value of 0.59% given in Table 7.

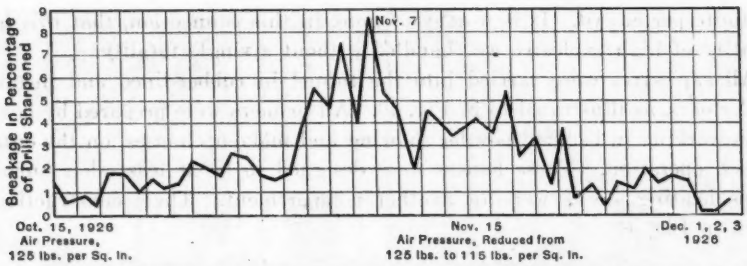


FIG. 42.—GRAPHIC RECORD OF BROKEN DRILL STEEL AT WEST PORTAL OPERATIONS, SHOWING RESULT OF SLIGHT DEPARTURE FROM PRESCRIBED METHODS.

Drill steel was hauled into the working places in special cars provided with compartments, as shown in Fig. 43, and a full supply of steel for one round was loaded in one car, taken in, used, and brought back to the steel shop for refitting. (For details of drill sharpening see Appendix A.)

TABLE 7.—RECORD OF BROKEN DRILL STEEL AT MILL CREEK SHAFT OPERATION, NEW CASCADE TUNNEL, SHOWING RESULTS THAT MAY BE OBTAINED WITH PRECISE REGULATION OF FORGING AND TEMPERING TEMPERATURES

Year and month	Number sharpened	Number broken	Broken drills in percentage of total sharpened
September, 1926	6 959	10	0.14
October, 1926	19 046	6	0.03
November, 1926	28 656	166	0.58
December, 1926	32 691	66	0.20
January, 1927	36 399	116	0.31
February, 1927	31 015	319	1.03
March, 1927	42 560	312	0.73
April, 1927	48 169	462	0.96
May, 1927	52 372	400	0.76
June, 1927	60 550	440	0.73
July, 1927	60 209	258	0.43
August, 1927	62 535	277	0.44
Total	481 161	2 832	0.59

DRILLS AND DRILLING LAYOUTS

Drilling in the various operations was done by many types of drills, produced by several different manufacturers. Great credit is due the American manufacturers of rock drills, who have produced what is perhaps, when considered from all angles, the most effective machine known to present-day construction. Fig. 44 is a view of drilling operations. Typical drilling and loading diagrams, which are largely self-explanatory, are shown in Fig. 45.

EXPLOSIVES

The next item of importance in tunneling operations, after drilling, is a proper explosive. In the Cascade Tunnel gelatine dynamite varying from 60 to 80% in strength, was used, in a total of 4 197 877 lb. In the tunnel 994 178 cu. yd. of rock were excavated, requiring an average of 4.2 lb. of

dynamite per cu. yd. It is worthy of note, in this connection, that this large quantity of high explosive was handled without a single fatality.

All explosives were carried into the tunnel in rubber-lined and insulated powder cars, as illustrated to in Fig. 46. All primers were prepared by powder fitters working in isolated powder thawing and make-up houses, on the outside at each operation. These houses were designed to be at least slow burning, as the heating stoves were in another compartment. Their construction is shown in Fig. 47.

Mucking

Because of the restricted space available the work of removing broken rock (commonly called "mucking") is the most difficult operation to equip properly with mechanical appliances. With modern rock drills and explosives, little difficulty need be expected with breaking the ground, but under present conditions hand labor for "mucking" on a long tunnel is not to be considered and loading machines that will work within the cross-section areas of drifts and full-sized railway tunnels are highly specialized machines.

All "mucking" in drifts was done by machines, such as illustrated in Figs. 48 and 49. The best record of a machine of this type is the loading of 32 cu. yd. of hard, coarsely broken granite in 1½ hours, including the time occupied in cleaning up the fly rock.

The "mucking" in the enlargement was done by rebuilt railway-type shovels operated by compressed air. The longitudinal and cross-sections of the general arrangement of these shovels are shown in Fig. 50. These were originally steam shovels mounted on two 4-wheel railroad trucks, which were removed and replaced with caterpillar treads. Their boilers were removed and replaced with air receivers.

The shovel engines were of the slide-valve type and in order to prevent freezing at the exhaust, the air receivers were equipped with eighteen direct-current, 220-volt, immersion heating units connected at 100 volts to prevent destruction by overheating. Each receiver was also equipped with an indicating thermometer, the indicating element being mounted in front of the shovel operator, together with a switch that controlled the heating elements, in order that the temperature could be kept within practical limits.

Cars and Switching

The success of all mucking operations in tunnels and drifts depends upon the speed with which cars can be switched behind the mucking machine. In the Cascade Tunnel the switching was accomplished by transferring the empty cars from one track to another with car transfer hoists. On the model of shovel used in the main tunnel these car transfer hoists were constructed on the rear of the shovel, as shown in Fig. 50, the switching operation being shown in plan diagrammatically in Fig. 51. For use in the drifts where mucking machines (Fig. 48) were used, special car transfer hoists, running on their own trucks, were designed and built. The design of these car trans-

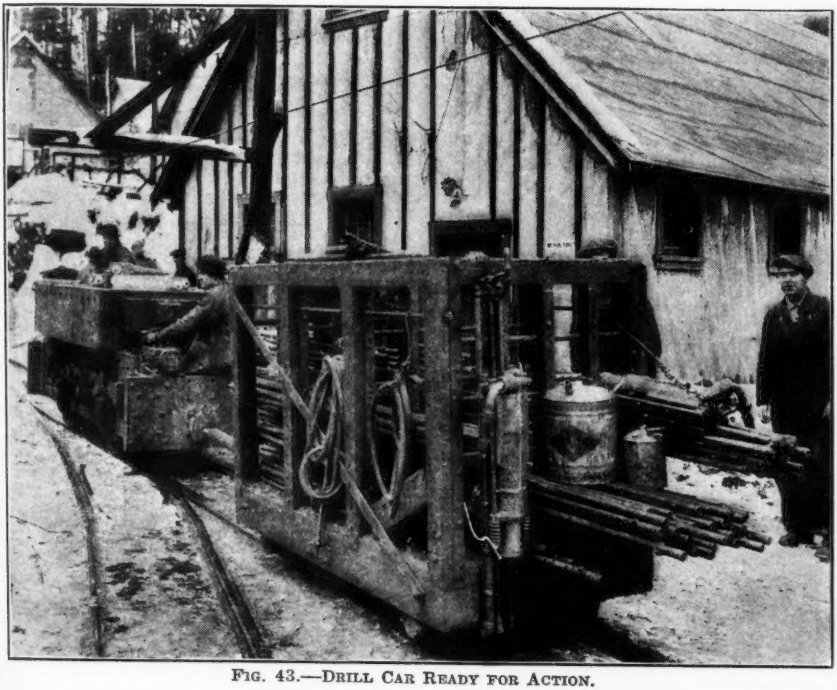


FIG. 43.—DRILL CAR READY FOR ACTION.

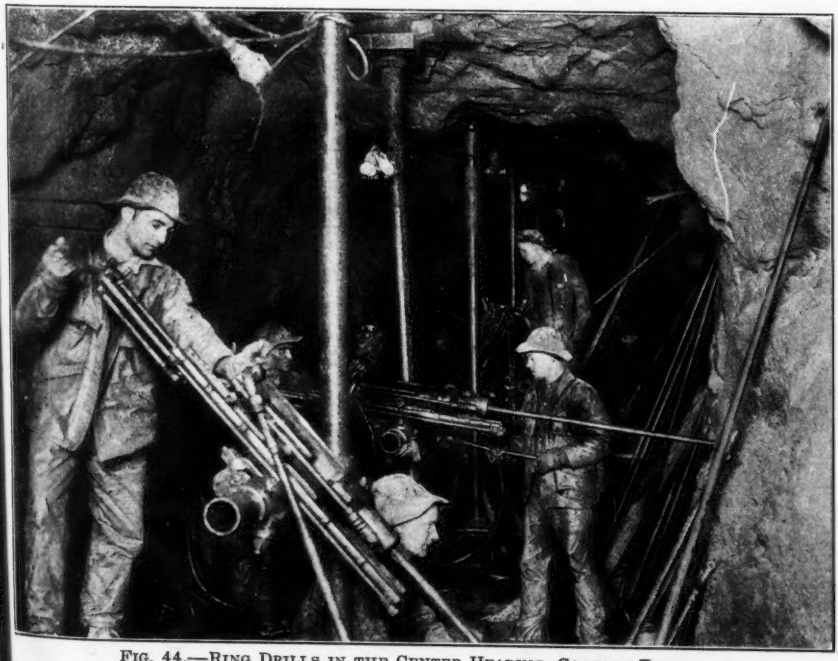
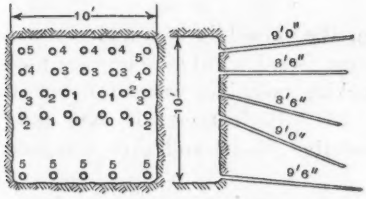
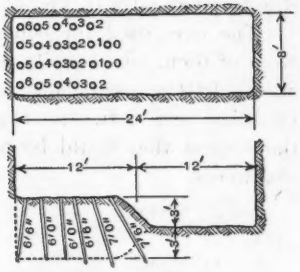


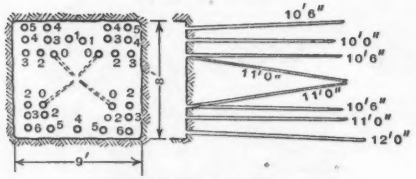
FIG. 44.—RING DRILLS IN THE CENTER HEADING, CASCADE TUNNEL.



EAST PORTAL CENTER HEADING
Average Total Length of Holes 237 Feet
Average Loading 259 Pounds of 60% Dynamite

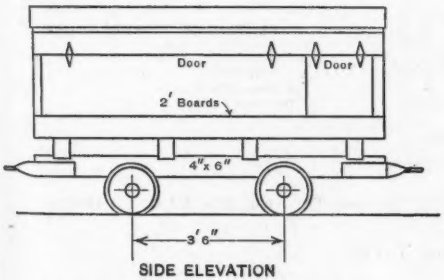


MILL CREEK SHAFT
Average Total Length of Holes 143 Feet
Average Loading 150 Pounds of 60% Dynamite

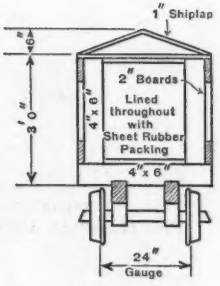


TYE RIVER INCLINE
Average Total Length of Holes 313 Feet
Average Loading 265 Pounds of 60% Dynamite

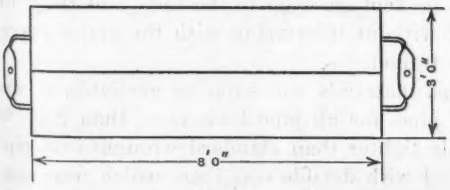
FIG. 45.—DRILLING AND LOADING DIAGRAMS.



SIDE ELEVATION



END VIEW



PLAN

FIG. 46.—PLAN AND ELEVATION OF CASCADE TUNNEL POWDER CAR.

fer hoists is shown in Fig. 52 and the switching layout is shown in plan diagrammatically in Fig. 53.

The cars used for conveying muck from the shovel in the main tunnel were of 6 cu. yd. capacity and 3-ft. track gauge (see Fig. 54). The cars used in the drifting operations for serving the mucking machines were of 50 cu. ft. capacity and 2-ft. track gauge (see Fig. 55). Both types of cars were the largest that could be used in their respective places and give adequate clearances.

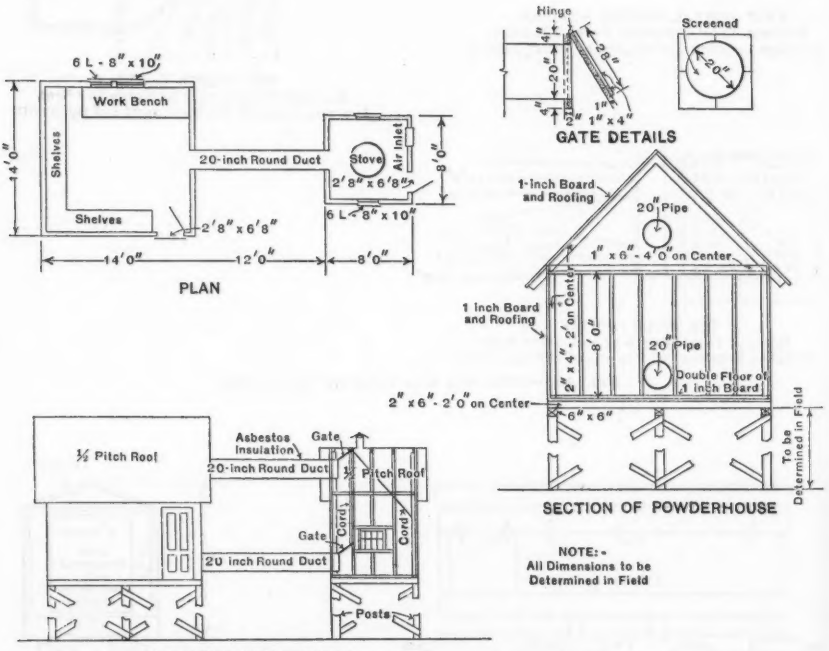


FIG. 47.—PLAN AND CROSS-SECTIONS OF POWDER THAWING AND MAKE-UP HOUSE.

PIPE LINES

The laying of pipe lines for compressed air and water is one of the lesser problems in tunnel operations, but still one of considerable magnitude, as the main air and water lines must be kept close up to the face, and the work of laying them must be performed without interfering with the major operation of drifting and enlarging the tunnel.

After careful studies of the pipe materials and supplies available, it was decided to use so-called "gas line" pipe, for all pipe lines more than 2 in. in diameter. This pipe is considerably lighter than standard wrought-iron pipe of equal diameter. It was connected with flexible couplings which were very easily applied, did not require any threading of the pipe, and permitted each length of pipe to be laid several degrees out of line. The use of this light pipe with flexible couplings was found to be entirely successful.

FIG. 48.—VIEW OF MUCKING SHOVEL USED ON THE CASCADE TUNNEL.

FIG. 49.—MUCKING MACHINE IN OPERATION IN THE PIONEER TUNNEL, TYE RIVER INCLINE, WEST PORTAL, SCENIC, WASH., MAY 7, 1926.

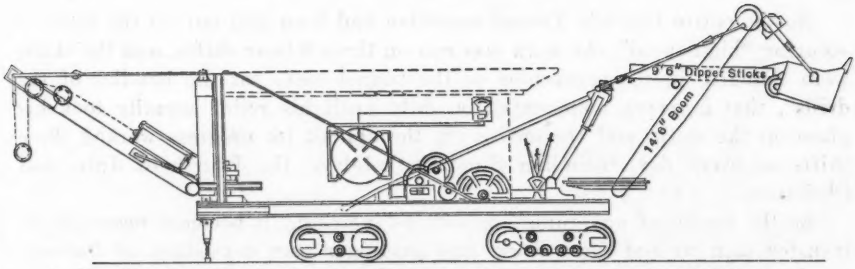


FIG. 50.—EXAMPLE OF RAILWAY TYPE OF SHOVEL REMODELED FOR TUNNELING.

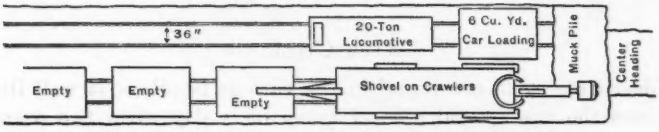


FIG. 51.—DIAGRAM OF SWITCHING IN FULL TUNNEL.

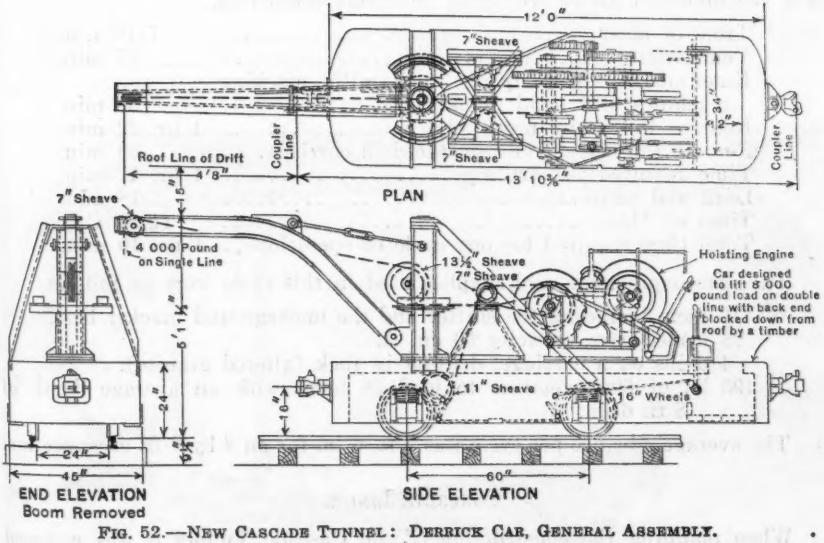


FIG. 52.—NEW CASCADE TUNNEL: DERRICK CAR, GENERAL ASSEMBLY.

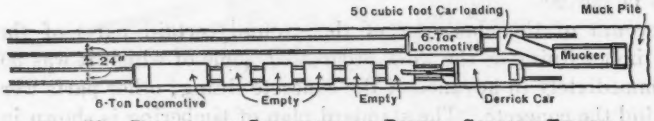


FIG. 53.—DIAGRAM OF SWITCHING IN DRIFTS, CASCADE TUNNEL.

Working Time

As the entire Cascade Tunnel operation had been laid out on the basis of securing "high speed", the work was run on three 8-hour shifts, and the shifts were changed, in the vernacular of the tunnel man, "at the handles of the drills", that is, every man stayed on duty until his relief actually took his place on the work, and the entire job throughout its progress worked three shifts on every day, including Sundays, holidays, the Fourth of July, and Christmas.

As the length of any tunnel operations increases, it becomes necessary to transfer men to and from work, and passenger cars consisting of flat-cars provided with a seat down the center on which two ranks of men could ride, sitting back to back, were constructed and used. These cars are shown in Fig. 56.

Time Cycle

The ideal time cycle per round in the various headings is well illustrated by that from the west portal pioneer taken on a day when five rounds were pulled and mucked in three shifts. The following tabulation of one cycle of operations at the Tye River Heading on August 25, 1926, represents what was accomplished under ordinarily favorable conditions.

Time of blast	7:10 A. M.
Ventilation	27 min.
Enter drift, clean up fly rock with mucking machines and hand muckers	38 min.
Remove the main muck pile	1 hr. 22 min.
Remove the mucker and set the drill carriage	38 min.
Time required for drilling	1 hr. 17 min.
Load and retire	18 min.
Time of blast	11:50 A. M.
Total time required for one cycle of operations	4 hr. 40 min.

The men, equipment, and supplies used in this cycle were as follows:

16 men, including the shifter and the underground mucker boss.
28 loaded cars, capacity 50 cu. ft.
4 drills on a carriage, drilling in rock (altered granite).
195 lb. of 60% gelatine to load 28 holes with an average depth of 8 ft. 6 in.

The average advance per cycle was 7 ft. 6 in. for an 8 by 9-ft. cross-section.

Concrete Lining

When planning the construction of the Cascade Tunnel it was assumed that concrete lining would be required throughout its length; this assumption proved to be correct.

On account of the character of the ground certain parts of the tunnel required timber lining as the work progressed, some of which it was possible to remove immediately in advance of the concrete lining; other parts were left in place behind the concrete. The standard plan of timbering is shown in Fig. 57 and the standard section of the concrete lining, in Fig. 58.

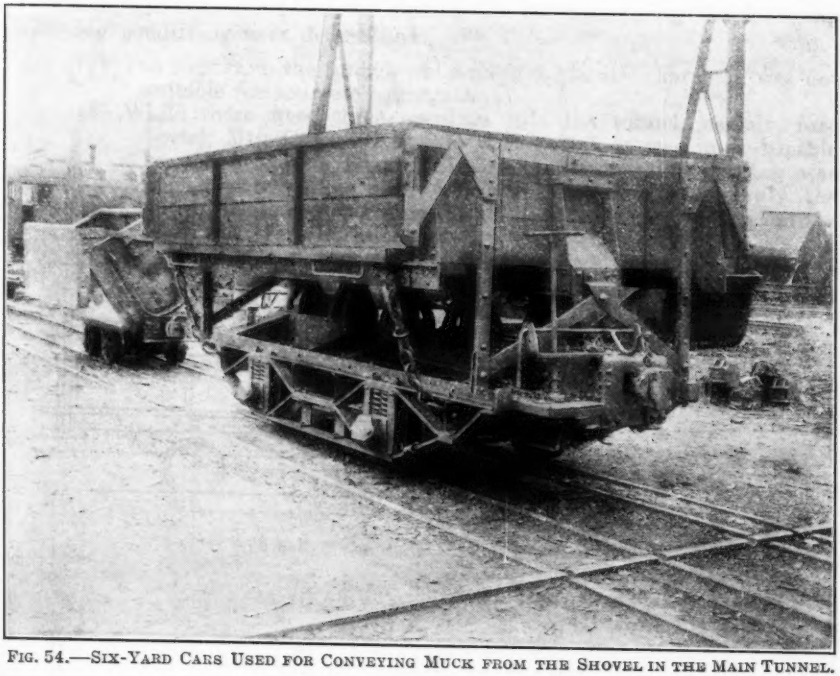


FIG. 54.—SIX-YARD CARS USED FOR CONVEYING MUCK FROM THE SHOVEL IN THE MAIN TUNNEL.

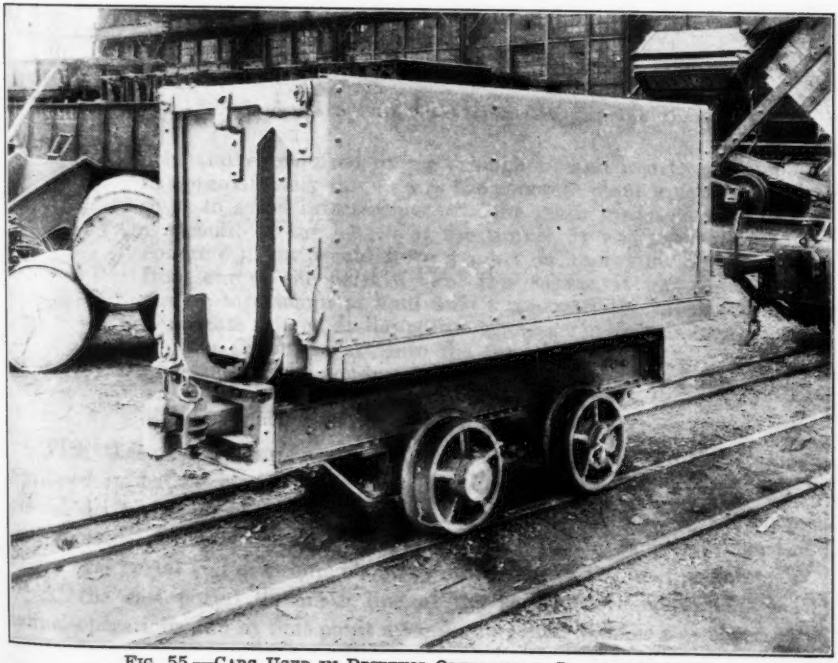
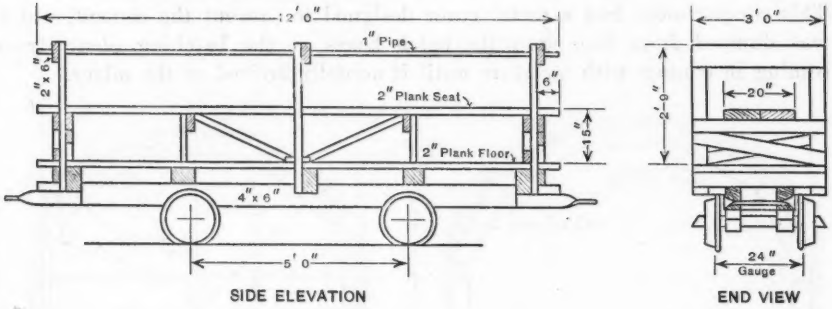


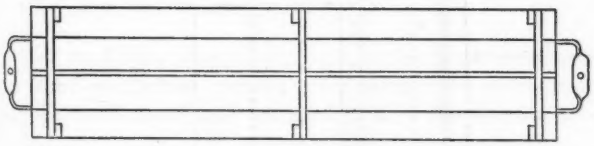
FIG. 55.—CARS USED IN DRIFTING OPERATIONS, CASCADE TUNNEL.

Before designing the plant for placing the concrete lining the following governing conditions were determined:

(*a*) The rock from the tunnel, while all of a granitic character, was not suitable for concrete aggregate.

(*b*) While there were three openings into the tunnel, namely, east portal, Mill Creek shaft, and west portal, it was impracticable to pour concrete from Mill Creek shaft as this opening was approximately three miles from the railroad. Consequently, the entire concreting operations would have to be carried on from the East and West Portals.





PLAN

FIG. 56.—ELEVATIONS AND PLAN, PASSENGER CAR, CASCADE TUNNEL.

(*c*) As the tunnel required lining throughout and was to be completed in approximately three years the concrete plant must be designed so as to avoid interference with the main tunnel operations.

(*d*) On account of the length of the tunnel, it was probable that the concrete lining would form a junction approximately four miles from one of the portals. For this reason, it was deemed inadvisable to attempt to haul mixed concrete, the alternative being to operate mixers at the points where concrete was being poured, hauling the aggregate into the tunnel dry.

(*e*) On account of the conduits required in both walls of the tunnel it was deemed necessary to pour the side walls in two lifts to expedite the placing of these conduits.

With these conditions in mind the Great Northern Railway Company equipped its ballast pit at Reiter, Wash., to produce concrete aggregate from glacial drift, that is, natural washed sand, washed gravel, and crushed gravel, and arranged to deliver this material in standard bottom hopper railroad cars at the east portal and at Scenic about 1½ miles west of the west portal.

At the east portal the main line of the railway was contiguous to the tunnel operations and at that point a sand and gravel bunker was constructed,

into which the railway cars unloaded by gravity. A similar bunker was constructed for the west portal, but as the main-line railway at the west end was 1½ miles from the tunnel portal the sand and gravel were rehandled over a 3-ft. gauge railroad to a second bunker at the tunnel portal. Both the sand and gravel bunkers were equipped with batching devices, using scale beams for sand batchers and volumetric batchers for the gravel. The batchers were discharged into batch boxes, four of which were carried on 3-ft. gauge flat cars. Each batch box was made with a vertical partition creating a compartment just large enough to hold the required quantity of cement for one batch. This compartment had a metal cover designed to prevent the cement, which was dumped from bags into the batch boxes at the batching plant, from coming in contact with moisture until it actually arrived at the mixer.

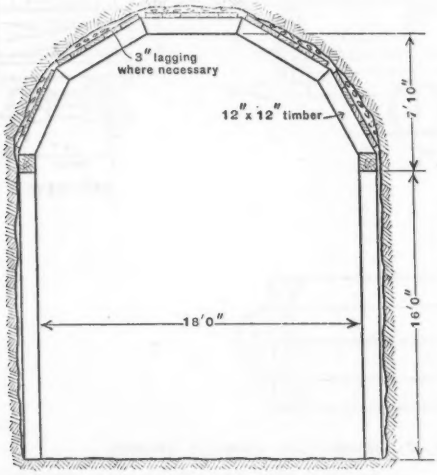


FIG. 57.—CASCADE TUNNEL: STANDARD TIMBERED TUNNEL SECTION.

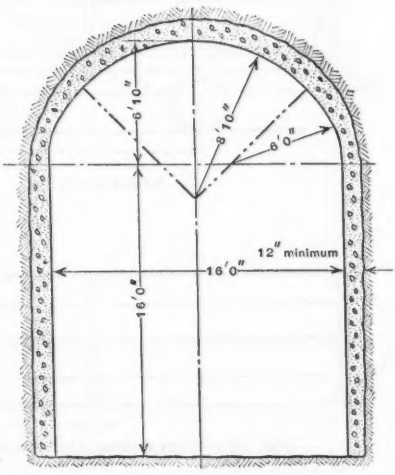


FIG. 58.—CASCADE TUNNEL: STANDARD CONCRETE LINING SECTION.

In general, the proportions of cement, sand, and gravel, used in the lining were 1 : 2 : 4, but these proportions were varied from time to time by the engineers to meet the varying characteristics of the aggregate. Practically all concrete had varying percentages of diatomaceous material added to improve its flowing qualities and the water-tightness of the lining.

A traveling steel form was designed[4] which permitted pouring of the side walls in two lifts and which completed 37½ ft. of tunnel at each move. The travelers for this form carried (in addition to the forms) hoisting apparatus, a 14-cu. ft. concrete mixer and air placer, together with various chutes and elevators, and permitted pouring of the lining up to the springing line of the arch by gravity, the arch itself being placed pneumatically. Cross-section drawings of these forms and their travelers are shown in Fig. 59.

Eight form travelers with their necessary equipment were used, three being operated from the west portal and five from the east portal.

[4] By and with the assistance of Blaw-Knox Co.

In laying out the concrete schedule for lining it was assumed that the forms should remain in place for 48 hours after pouring the last concrete in them. As the work progressed, this time was gradually shortened until move-

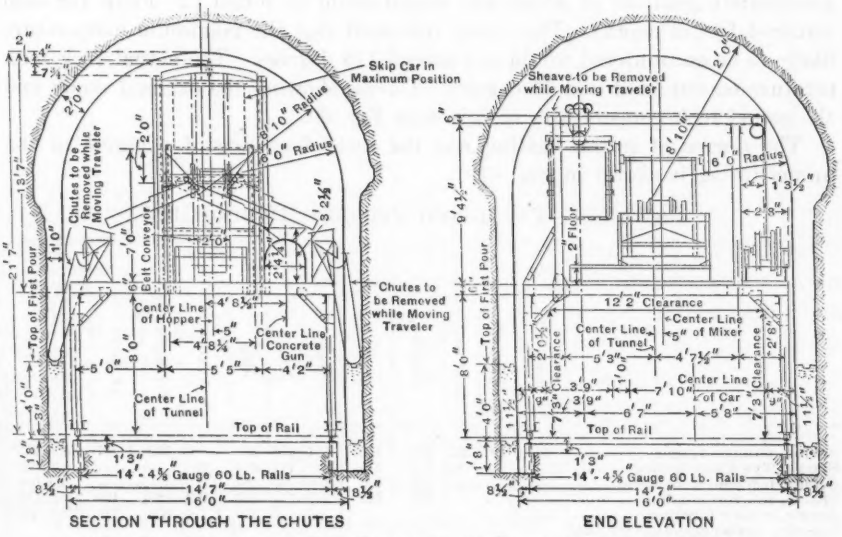


FIG. 59.—SECTION AND END ELEVATION OF TRAVELER, CASCADE TUNNEL.

ments of forms were made in 12 hours after the placing of the last concrete in that particular section. On numerous occasions, however, forms were struck in 10 hours and the records show the shortest time in which a form was

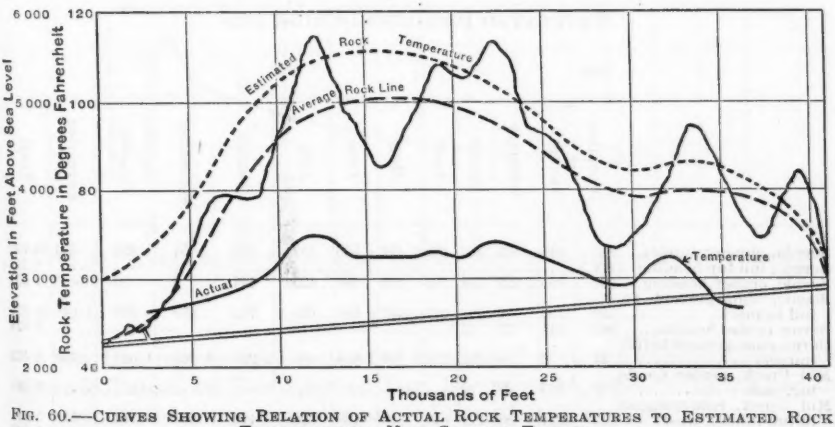


FIG. 60.—CURVES SHOWING RELATION OF ACTUAL ROCK TEMPERATURES TO ESTIMATED ROCK TEMPERATURES, NEW CASCADE TUNNEL.

struck to have been 8 hours after the final placing of the concrete. In no instance was there any evidence of damage to the concrete. For a detailed discussion of air placing of concrete in the arch ring, see Appendix B.

Miscellaneous Data

Before proceeding with the Cascade Tunnel a study was made of the possible rock temperatures that might be encountered, using the very highest geo-thermic gradient of which any record could be found (2° Fahr. for each hundred feet of depth). This study indicated that the maximum temperature likely to be encountered would not exceed 115 degrees. The actual rock temperature encountered was 75 degrees. Curves showing the original study and the actual rock temperatures are given in Fig. 60.

The degree of initial heating and the rate of cooling of concrete in the tunnel lining is shown in Fig. 61.

TABLE 8.—Tabulated Progress During 1926

	1925	1926											
	December	January	February	March	April	May	June	July	August	September	October	November	Total
Scenic, pioneer tunnel	40	190	224	226	100	424	458	485	63	512	1 157	952	8 275*
Scenic, Tye River incline	.....	40	82	178	376	651	717	676	937	....	.....	....	3 657
Scenic, top center heading	.....	....	....	193	162	158	205	282	...	....	.....	....	1 000
Scenic, full top heading	.....	....	...	100	142	129	177	191	224	195	496	373	2 036
Scenic, center heading	.....	....	....	....	.....	56	0	570	1 196	659	216	926	3 623
Scenic, enlargement to full tunnel	.....	....	....		.....	....	....	....	415	348	451	597	1 811
Berne, center heading	14	111	335	476	488	657	709	758	665	954	893	938	6 998
Berne, enlargement to full tunnel	.....	...	...	.....	.....	...	...	....	.....	....	42	62	104
Mill Creek, shaft	.....	....	61	106½	122½	78	158	133	.....	....	.....	....	659
Mill Creek, center heading, east	.....	....	....	.....	.....	....	....	.....	.....	159	707	648	1 514
Mill Creek, enlargement to full tunnel, west	.....	....	....	.....	.....	....	....	.....	.....	114	144	200	458
Mill Creek, pioneer tunnel	.....	....	....	....	.....	....	....	.....	....	....	66	620	686

Tabulated Progress During 1927

	1926	1927											
	December	January	February	March	April	May	June	July	August	September	October	November	Total
Scenic, pioneer tunnel	819	844	579	477	273	414	556	647	582	454	491	621	15 032
Scenic, full top heading	179	....	.....	....	....	.....	.....	.....	.....	.....	.....	.....	2 215
Scenic, center heading	992	990	1 222	723	283	218	606	625	144	0	20	105	9 551
Scenic, enlargement to full tunnel†	532	720	700	786	930	697	586	623	792	575	351	1 048	10 151
Berne, center heading	865	845	723	93	....	.....	.....	.....	.....	.....	.....	.....	9 524
Berne, enlargement to full tunnel	47	0	0	343	776	1 008	1 064	900	1 220	1 160	1 040	760	8 422
Mill Creek, center heading, east	793	765	63	10	....	.....	.....	.....	.....	.....	.....	.....	3 145
Mill Creek, enlargement to full tunnel, west‡	220	244	83	112	548	696	188	556	472	564	484	516	5 141
Mill Creek, pioneer tunnel	745	752	708	607	854	625	756	579	630	319	236	410	7 907
Mill Creek, center heading, west	17	0	663	624	405	796	613	714	545	768	526	527	6 198
Scenic, concrete lining	108½	0	0	0	0	945	1 163	1 161½	1 350	1 387½	787½	0	6 903
Berne, concrete lining	.....	....	.....	....	....	.....	.....	356½	1 162½	1 425	1 500	1 500	5 944

*Represents total of pioneer tunnel and incline.

†Includes enlargement at Nos. 5 and 8.

‡Does not include 69 ft. east of shaft.

TABLE 8.—(*Continued*)

TABULATED PROGRESS DURING 1928

	1927	1928											
	December	January	February	March	April	May	June	July	August	September	October	November	Total
Scenic, pioneer tunnel...	558	543	541	522	537	7	.....	.....	.....	.....	.....	.....	17 740
Scenic, center heading..	482	481	517	541	287	0	.....	.....	.....	.....	.....	.....	11 859
Scenic, bottom heading.	.....	14	100	68	0	187	235	488	308	.....	.....	.....	1 400
Scenic, enlargement to full tunnel‡.....	839	532	668	396	388	480	821	1 075	443	398	762	765	17 718
Berne, enlargement to full tunnel.....	476	660	878	742	776	720	.....	.....	.....	.....	.....	.....	12 674
Mill Creek, enlargement to full tunnel, west‖....	512	480	368	438	412	409	243	136	303	679	814	726	10 730
Mill Creek, pioneer tunnel.....	708	499	647	442	325	.....	.....	.....	.....	.....	.....	.....	10 528
Mill Creek, center heading, west.....	318	54	42	259	408	107	436	524	415	.....	.....	.....	8 861
Mill Creek, bottom heading.....	.....	.....	.....	.....	.....	253	383	84	291	.....	.....	.....	1 011
Scenic, concrete lining..	750	1 143½	1 106½	675	1 050	712½	0	37½	901	712	976	1 311	16 278
Berne, concrete lining...	1 500	1 049	0	976½	1 199½	1 237½	1 201½	2 137½	2 101	2 174	1 124	1 687½	22 332

‡ Includes enlargement at Nos. 10, 10*B*, 11, 12, 12*A*, and 12*B*, and an allowance of 60 ft. for top-heading enlargement between Nos. 14 and 15.

‖ Includes 69 ft. east of shaft and enlargement at Nos. 14 and 15.

In Figs. 62 and 63 are shown the total man-days as compared with cubic yards of excavation and cubic yards of concrete, respectively.

The progress in all the various operations is shown in Table 8. The wage scale is shown in Table 9.

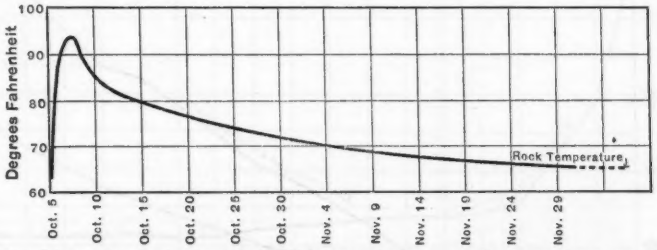


FIG. 61.—CURVE SHOWING CONCRETE TEMPERATURES FROM TIME OF POUR TO TIME IT ASSUMED TEMPERATURE OF ADJACENT ROCK.

CONCLUSION

The Contractors were instructed to proceed with the construction of the Cascade Tunnel on Thanksgiving Day, November 26, 1925. Clearing and the construction of camps were commenced immediately and had advanced so far by December 14, that the approach cut for the west portal of the pioneer tunnel was started on that date. The approach cut for the center heading at the east portal was started on December 29, 1925, and the sinking of Mill Creek shaft on January 30, 1926.

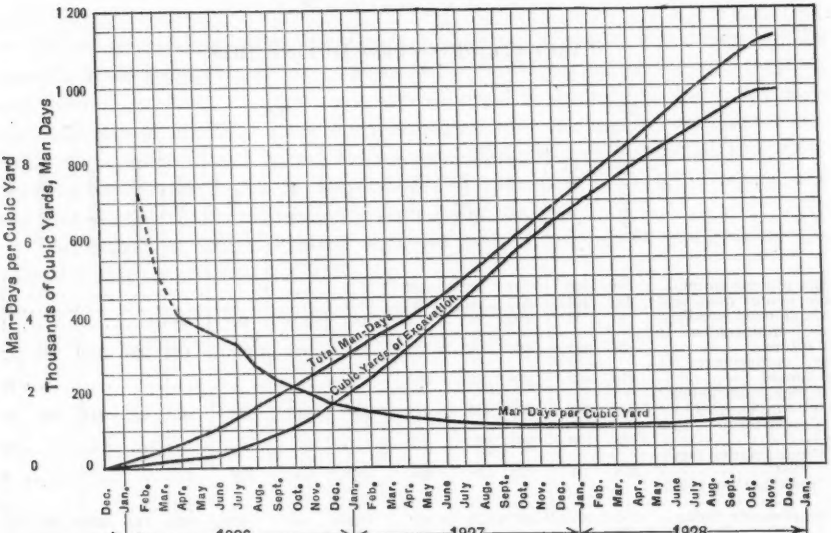


FIG. 62.—RELATION BETWEEN MAN-DAYS EMPLOYED ON EXCAVATION AND ACTUAL NUMBER OF CUBIC YARDS MOVED.

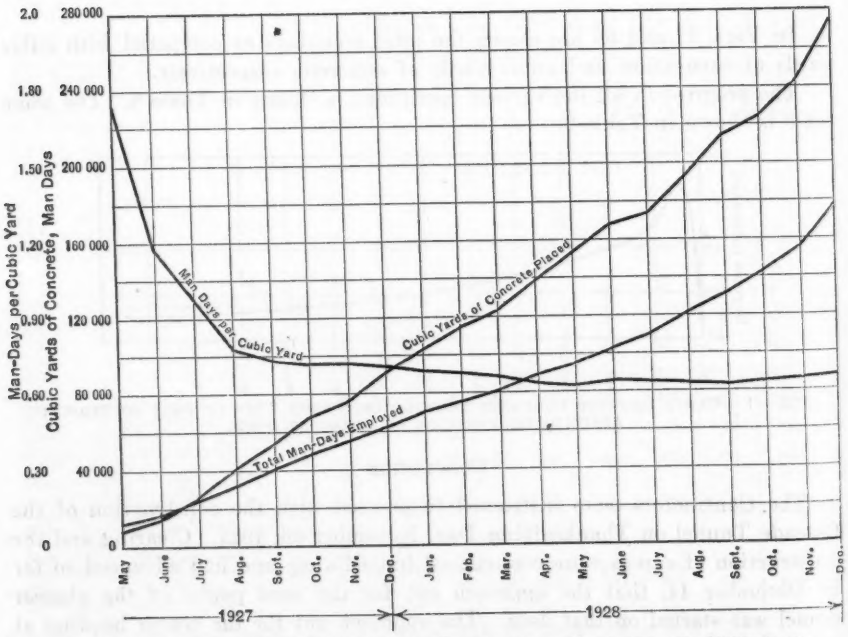


FIG. 63.—RELATION BETWEEN MAN-DAYS EMPLOYED ON CONCRETE AND NUMBER OF CUBIC YARDS PLACED.

TABLE 9.—WAGES PAID BY CONTRACTOR IN THE 8-MILE CASCADE TUNNEL, APRIL 1, 1926

Accountant, Camp	$150 or $175 and board*
Axeman	$0.56¼
Baker	$90 or $125 and board
Barber (Headquarters)	$125 and board + 10% of business
Blacksmith	$0.80
Blacksmith helper	$0.55 or $0.60
Brakeman	$0.60
Bull Cook, Foreman	$75 and board
Bull Cook	$70 and board
Cageman	$0.60
Camp flunky	$60 and board
Camp flunky, Head	$65 and board
Carpenter	$0.75
Chief Clerk (Headquarters)	$200 and board
Clerk	$100, or $125 and board
Cook	$125, or $200 and board
Cook, Assistant	$75, or $125 and board
Deputy Sheriff	$150 and board
Dishwasher	$60 and board
Draftsman (Headquarters)	$150 and board
Driller	$0.80
Driller, helper	$0.65
Drill doctor	$0.80
Drill sharpener	$0.80
Electrician	$0.80
Electrician, helper	$0.60
Faller	$0.62½
Fireman	$0.62½
Foreman, Carpenter	$1.00
Foreman, Clearing	$0.80-1.00
Foreman, Concrete	$0.75-1.00
Foreman, General	$250 and board
Foreman, Labor	$0.75
Foreman, Mechanical	$1.00
Foreman, Rigger	$1.00
Foreman, Steam Shovel	$1.75 and board (outside)
Handyman, Surface	$0.60
Handyman, Underground	$0.70
Helper, Kitchen	$65, or $75 and board
Hoist Operator	$0.80
Inspector, Safety (Headquarters)	$200 and board
Labor Agent	$150
Laborer, Surface	$0.50
Laborer, Underground	$0.60
Locomotive Operator	$0.75
Mechanic, Master (Headquarters)	$300 and board
Mechanic	$0.80
Mucker Operator	$0.80
Plant Operator	$0.75
Powder Fitter	$0.80
Pumpman	$0.60
Rigger	$0.75
Saw Filer*	$0.68¾-0.75†
Shifter	$0.90
Stenographer	$100, or $125 and board
Storekeeper, General (Headquarters)	$150, or $175 and board
Storekeeper, Assistant (Headquarters)	$100, or $125 and board
Teamster	$75 and board
Timberman	$0.75
Timekeeper	$100, or $125 and board
Top Man (shaft only)	$0.60
Tractor Operator, 2-Ton	$125 and board
Tractor Operator, 10-Ton	$150 and board
Truck Driver	$100 and board
Team (rented)	$30, or $40 and board
Walker	$200, or $250 and board
Watchman	$60 and board
Waitress	$45 and board

* Board and sleeping accommodation = $1.35 per day. Men receiving rates, including board, were paid an allowance of $25 per month when boarding at home.

† This classification expired April 30, 1926.

The center heading, between the east portal and Mill Creek shaft, was holed through March 4, 1927, eighty-eight days ahead of schedule. The elapsed time on this operation was 430 days, during which 12 733 ft. of 10 by 10-ft. drift was excavated from two faces at an average rate of 14.8 ft. per day at each face, attaining a maximum month's progress at one face of 954 ft.

The pioneer tunnel between the west portal and Mill Creek shaft was holed through May 1, 1928, the final shot being fired from Washington, D. C., by President Coolidge. This operation was completed fifteen days ahead of schedule in an elapsed time of 867 days, during which 28 268 ft. of 8 by 9-ft. drift was driven from two faces, together with 21 cross-cuts, each 100 ft. long, or a total length of 30 368 ft., at an average rate of 17.5 ft. per day at each face, attaining a maximum month's progress at one face of 1 157 ft.

The enlargement between the east portal and Mill Creek shaft was completed 150 days ahead of schedule in an elapsed time of 452 days, during which 12 733 ft. of tunnel was enlarged to full section from one face, at an average rate per day of 28.2 ft., attaining a maximum month's progress at one face of 1 220 ft.

The enlargement between the west portal and Mill Creek shaft was completed December 8, 1929, eight days behind schedule, thus completing the excavation for the Cascade Tunnel, from which was removed 994 178 cu. yd. of rock.

Concrete lining was commenced from the west portal, April 27, 1927, and from the east portal, July 22, 1927, and was completed December 24, 1928, sixteen days after the completion of the excavation. The entire length of the tunnel was lined, a distance of 41 152 ft., at an average rate of 75.5 ft. per day, attaining a maximum month's progress of 3 002 ft. The total quantity of concrete placed in lining the tunnel was 275 218 cu. yd.

Track-laying and ballasting were commenced December 25, 1928, and the first scheduled train ran through the tunnel from east to west on January 12, 1929, 3 years and 47 days after the Contractors had been instructed to proceed with the work. From the date of instructions to proceed, to the date of the first scheduled train, the Cascade Tunnel was completed at an average rate of 36 ft. per day.

The opening of the tunnel was attended by a notable group of statesmen, railway executives, and engineers. To commemorate the occasion a remarkable radio broadcasting hook-up was arranged through thirty-two stations, over which speeches were made by President Hoover, from Washington; W. W. Atterbury, M. Am. Soc. C. E., President of the Pennsylvania Railroad, from New York; and J. B. Campbell, of the Interstate Commerce Commission, from Washington. When the first passenger train entered the tunnel the fact was announced by Ralph Budd, M. Am. Soc. C. E., President of the Great Northern Railway Company, speaking from the train itself. Thus was placed in service the longest railroad tunnel in the Western Hemisphere and the fifth longest railroad tunnel in the world.

APPENDIX A

DRILL-STEEL SHARPENING PRACTISE AT WEST PORTAL, NEW CASCADE TUNNEL

By H. J. King,[5] M. Am. Soc. C. E.

Description of Shop and Equipment.—The steel shop at the west portal was located between the 24-in. gauge service track leading to the pioneer tunnel and the 36-in. gauge track serving the main tunnel. A 24-in. gauge track leading from a switch in the pioneer track extended completely through the shop and beyond it into the machine and blacksmith shop located in the same building. The shop room, 32 ft. wide and 60 ft. long, was arranged with a view to minimizing the labor of handling steel in and out of the building and, at the same time, to give ample room for storage of bar stock and of made-up steel.

The equipment consisted of the following (see Figs. 64 and 65):

Three oil furnaces, equipped with pyrometer control, thermo-couplers, control-boxes, and magnetic indicators.
Two driller sharpeners.
One pedestal grinder.
One 250-lb. anvil.
One 3 000-gal. steel oil-storage tank (elevated).
One 1 500-gal. steel tank mounted on 3-ft. gauge, 6-yd. car truck (for transporting oil).
One 3 by 2 by 4 duplex pump (for handling fuel oil from tank car to storage tank).
Four platform trucks.
Storage racks for made-up steel.
Storage racks for bar stock (under shop).
Two quenching tanks (wood) for water (bits).
One quenching tank (steel) for oil (shanks).

Cutting-Bars.—Bars of 1¼-in. round, hollow steel, containing 0.78 to 0.85% of carbon, in lengths averaging 22 ft., were used for all primary drilling. A 1-in. hexagon steel of the same carbon content was used for stoper and jack-hammer steel. Bars were cut into the desired lengths by nicking them on opposite sides with a track chisel or cold cut. Then they were laid across the hub of an old 6-yd. car wheel and struck a smart blow with a 16-lb. hammer. Ragged ends of the bars on the shank end were then ground down to a square striking surface and properly chamfered.

Forging and Tempering New Bits and Shanks.—For forging shanks the pyrometer control was set at from 1 900° to 2 000° Fahr. About thirty bars of the same length were placed in the furnace and slowly brought to this temperature. Especial care was taken to avoid "soaking" the steel at high temperature. Only two or three steels were allowed to reach forging temperature at the same time, as oxidation takes place rapidly above 1 500° to 1 600° Fahr., with very harmful effects on the steel.

[5] Supt., A. Guthrie & Co., Inc., Portland, Ore.

After the bars were heated they were removed from the end of the combustion chamber farthest from the burner, placed in the sharpening machine, and upset sufficiently to give a bulb of metal large enough to form the shanks. The actual forging took place at temperatures between 1 800° and 1 900° Fahr. The steel must not be worked below a cherry red heat.

When the entire lot of bars had been "bulked" they were replaced in the furnace and heated as before. The bulb die was removed from the sharpener and the shanking die substituted, after which the bulbs were forged to exact shank dimensions. The same methods and precautions were used for this heat as for the preceding one.

After the shanks were forged they were allowed to cool to well below a black heat. They were then reheated in the furnace with the pyrometer control set at 1 600° Fahr. and the hearth opening as wide as possible. The entire shank up to a point at least 2 in. beyond the lugs was brought to a temperature of 1 500° Fahr., tested on the magnetic indicator to prove they had been heated past the point of decalesence, and quenched in the oil bath.

Bits were forged in exactly the same manner except for the substitution of appropriate dies and dollies. The process of hardening the bits differed from the treatment of the shanks only in a slightly lower temperature (pyrometer set at 1 550° to 1 600° Fahr., with a 2-in. hearth opening). The steels attained a temperature of about 1 450° Fahr., and were quenched in clear, cold, running water.

Reconditioning Steel.—A round of steel, as it was returned from the tunnel to the shop for re-sharpening, consisted of from 24 to 36 sets of steel, varying in length by 2 ft., from 3 ft. to 13 ft. long, or 144 to 216 bars. These were examined both for broken bits and shanks, and if any were found, they were set aside for measurement and for cutting down to shorter lengths. Bent steel was similarly taken from the car for straightening. The remainder of the carload was placed on two platform trucks, each length being separated by wooden strips placed between layers. The trucks were wheeled to Nos. 1 and 3 Furnaces and sharpened. As steels were sharpened they were replaced on empty trucks and, subsequently, taken to No. 2 Furnace, reheated, and tempered. As truck loads of reconditioned steel accumulated, they were wheeled to the storage racks to await loading on the steel cars for further use underground.

As drill steel has a limited life and can be expected to begin breaking rapidly as the life limit is approached, it is imperative, in order to avoid concentrated breakage, that new steel be introduced into the stock regularly. Consequently, it was the practice to make up a small number of new drills in the longer lengths each day. Breakage, particularly shank breakage, is more pronounced in the longer lengths in steel of the same age. This is caused by the more serious effect of machines being out of line when using longer steel. In the comparatively easy drilling rock in the Cascade Tunnel very little bit breakage was experienced. As crystallization almost always occurred immediately at or below the shanks long before much change took place in the structure of the steel forming the body of the bar, the longer ones were cut down as they broke at the shank, to the next shorter length. Con-

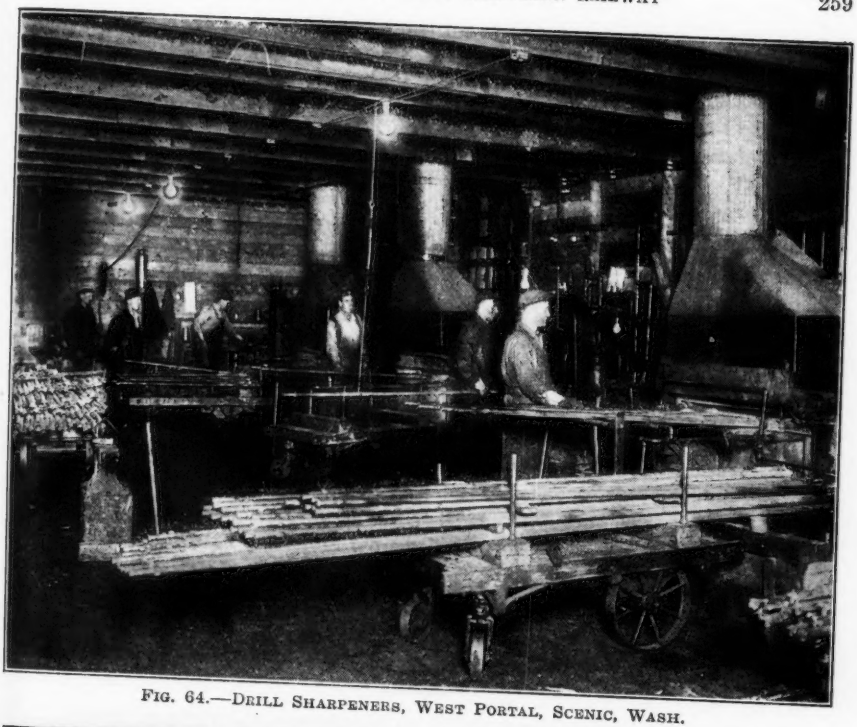


FIG. 64.—DRILL SHARPENERS, WEST PORTAL, SCENIC, WASH.

FIG. 65.—DRILL SHARPENERS AND FURNACE, SCENIC, WASH.

sequently, as time went on large quantities of the shorter lengths accumulated, particularly starters and 5-ft. steels. By the time a steel had been successfully cut down from 11 or 13 ft. to 5 or 3 ft., most of its useful life was gone. Consequently, actual breakage in the shorter lengths was greatest.

Where automatic heat control is used, as it was on the Cascade Tunnel, only two causes contribute greatly to premature breakage of steel. The first of these is the action of machines running out of line. Breakage from this cause cannot be attributed to faulty shop practice, and the remedy for it lies solely in the hands of those responsible for the operation of the drills. The second cause is due to using a drill with a pitted hammer face, or a face so badly worn down as to permit part or all of the hammer blow to be taken up by the lugs instead of the end of the steel. This is the largest single cause of premature shank breakage. An examination readily shows when this condition is present, and it is the function of the steel shop to remedy it by reshanking.

Selection of Personnel.—Experience at the Cascade Tunnel indicates that the old-time blacksmith or steel sharpener is often thoroughly saturated with worn-out theories and prejudice against any method or theory other than the one in which he believes. Consequently, it was found better to break in young, intelligent men with sufficient educational background to enable them to understand the theory of heat treatment.

APPENDIX B

PNEUMATIC CONCRETE PLACING PRACTISE IN THE NEW CASCADE TUNNEL

By H. J. King,[6] M. Am. Soc. C. E.

Description of Apparatus.—The pneumatic concrete placing machine used on the Cascade Tunnel (see Figs. 66 and 67) consists essentially of a horizontal steel cylinder, provided with an air-operated drop door through which the machine is charged, and an outlet opening recessed below the bottom of the forward end of the cylinder connected by means of flanges to the discharge line. A steel worm keyed to a heavy shaft is mounted in bearings in the ends of the cylinder at a sufficient height to permit the edge of the worm barely to clear the steel liner which protects the bottom side of the cylinder from abrasion. The worm is driven by a small motor mounted near the rear (or operator's) end, the speed being regulated by a train of gears in an oil-filled transmission case between the motor and the cylinder. The air inlet is situated at the rear end of the machine between the worm-bearing and the floor of the cylinder. A small air cylinder actuates the charging door, and the control of air supply to the cylinder is effected by a quick-acting valve which, together with the motor control, is mounted at the operator's end of the machine.

[6] Supt., A. Guthrie & Co., Inc., Portland, Ore.

Operation—First, the charging door is opened by the application of air to the door cylinder, and the proper volume of previously mixed concrete is allowed to flow into the cylinder. Before shutting the door, it is necessary to remove any concrete that may adhere to the upper side so as to prevent the escape of air between the door and the round rubber gasket ring upon which it is seated. This is best done by tapping the main air supply line and connecting a short length of ½-in. air hose fitted with a trigger nozzle. A quick circular motion around the edge of the door cleans it effectively with little or no loss of time. The door is then closed by reversing the piston travel in the cylinder. The worm motor is then started. After the worm has made two or three revolutions, the air-control valve is opened. As the air pressure builds up inside the cylinder, the concrete is moved forward by the worm and is finally forced through the outlet into the discharge line. No hard and fast rule can be set for the length of operation of the worm beyond stating that "the shorter the better". When they feel the movement of concrete start in the discharge line, some operators stop the worm for a few seconds, and then give it a turn or two just before shutting off the air. It should be borne in mind that the function of the worm is to deliver the concrete to the discharge recess in sufficient quantity to keep the recess full, and thus prevent the excessive use of air.

In theory, if the concrete is delivered to the discharge recess at exactly the same rate at which it is forced through the discharge line, it should move out as a solid plug. Owing to the fact that as the batch begins to leave the cylinder, the rate of delivery lessens, and also that there is a considerable loss of air through the concrete, the batch tends to separate as it moves along the discharge line, its velocity increasing as the dispersion becomes greater. It should be thoroughly impressed on all operators that they should try for minimum dispersion and that the continued feeding of air to the cylinder after the bulk of the batch is discharged is the principal cause of wear in discharge hose, bends, and nipples. It is the last of the batch which, moving at a very high velocity, impinges upon the walls of the discharge line, particularly at the bends which causes a large part of the wear. There is probably no class of construction equipment in the operation of which greater dependence must be placed upon the "feel" of the machine than this type of gun. Minute differences in the quantity of water in the mix, and slight variations in degree of "harshness" of the aggregate, exert a profound influence on the operation of the machine; consequently, every effort should be made to provide a uniform mix. The addition of diatomaceous material to harsh aggregate materially helps in the operation of the machine, but experience in the Cascade Tunnel indicated that the excessive use of this material involved a penalty in the increased difficulty in making the mixture "stand" in the key of the arch. However, its moderate use with all but the smoothest aggregates is recommended.

Provided suitable aggregate is used, with proper measures to overcome any harshness, concrete can be placed with this machine dry enough to satisfy the most drastic specifications. The disadvantage of trying to handle sloppy concrete in the top of a tunnel arch generally results in a dry mix being used by preference.

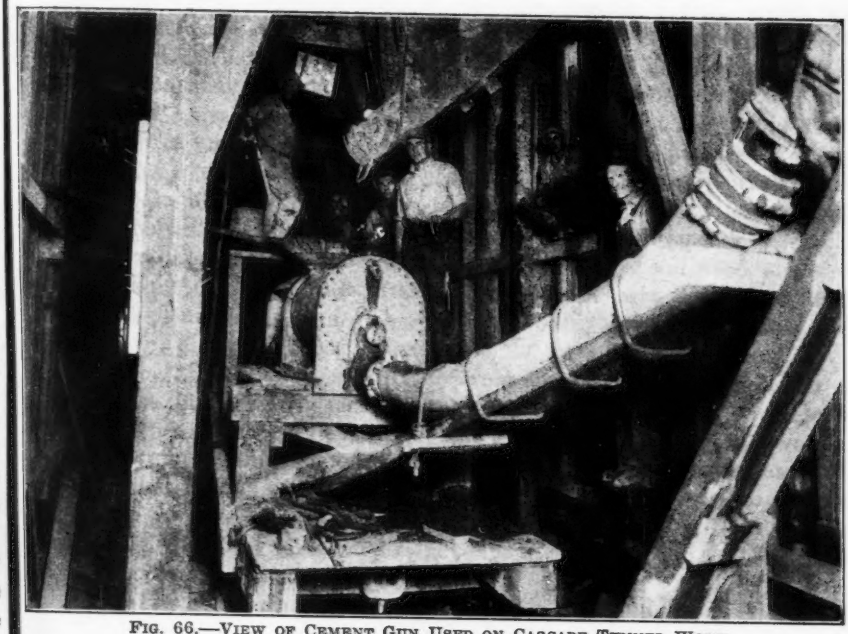


FIG. 66.—VIEW OF CEMENT GUN USED ON CASCADE TUNNEL WORK.

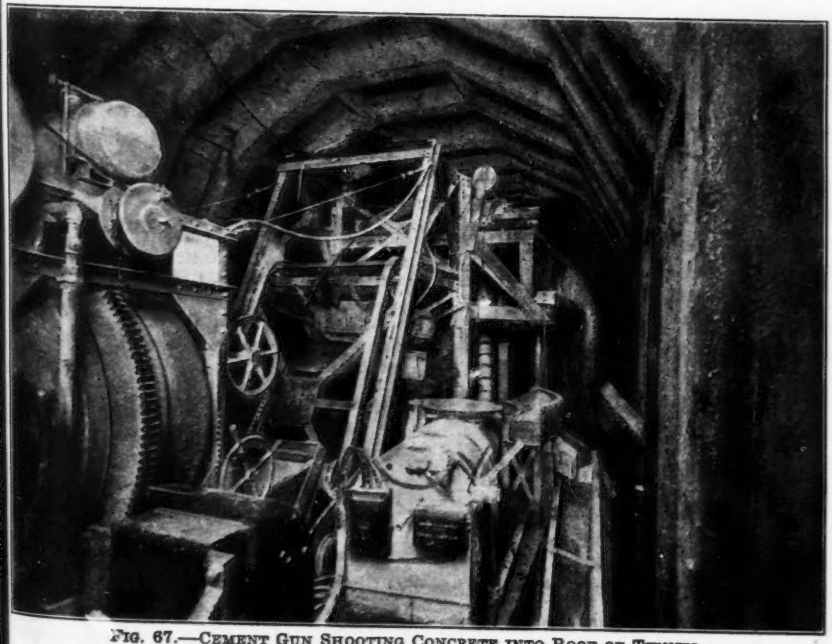


FIG. 67.—CEMENT GUN SHOOTING CONCRETE INTO ROOF OF TUNNEL.

Aggregate—With concrete guns of this type, having a 6-in. outlet, the aggregate should not exceed 2 in. in greatest dimension. If the percentage of fractured material is great this should be reduced to 1½ in. Slabby pebbles and thin elongated pieces of rock are very difficult to pass through the machine, and aggregate containing very many of these should be avoided.

Fairly well-rounded, well-graded washed gravel should be procured if available, even at a considerably higher cost. The loss of time resulting from "plugs" in the discharge line is lessened materially, if not altogether avoided, by the combination of this class of coarse aggregate with smooth, workable sand. Great care should be taken to prevent the small pieces of wood, pieces of gunny sacking, etc. (which are often used to prevent leaks in railroad cars) from being placed in the gun. These will invariably cause plugs. This also holds true for pieces of caked cement.

Discharge Lines—As a general rule, avoid abrupt changes in direction of the discharge line. Elbows should never be used. The more nearly straight the discharge line is, the greater will be the ease and certainty of operation, and the wear on the line is infinitely less. If changes in direction are unavoidable use long sweeping bends. Wear is greatest on the bottom of straight runs and on the concave side of bends; consequently, to insure uniform wear, a quarter turn should be taken on straight pipe occasionally and on hose bends after every pour of, say, 100 yd.

The makers of the machine recommend a specially built wire-wound rubber hose for discharge line. Experience at the Cascade Tunnel has proved beyond question that, for straight runs, wrought-iron pipe, extra heavy, will give considerably better service. Continued observation has indicated that the standard wire-wound hose gave reasonably good service until the soft, line-rubber, interior tubing was worn through. After this occurred wear was extremely rapid and a blow-out occurred within about an hour. When the wear penetrated to the spiral wire reinforcing it was only a question of a few batches until a plug occurred at the point of greatest wear. This was caused by the wire cutting in two, thus separating from the cotton hose wall and snarling inside the tube. With this in mind, it occurred to the writer that since no blow-outs had been observed, which could be traced to lack of wall strength, the wire reinforcing should be abandoned and its cost put into additional soft, line-rubber tubing. Consequently, a hose of the following characteristics was specially made up at a price considerably below that of the wire-wound product:

Weight, cotton fabric	32 oz.
Number of plies	8
Thickness of special abrasion-resisting rubber tubing	¾ in.
Length	5 ft. and 10 ft.
Diameter, internal	6 in.
Cost (f. o. b. Scenic) of one 10-ft. length, including nipples and clamps	$101.50

Thereafter, two lengths of this hose were used on each machine, the lower connecting directly to the gun was 5 ft. long, and the upper connecting to

the inclined discharge pipe was 10 ft. long. This combination gave the necessary flexibility of maneuver to that section of the line immediately above the form itself.

Due to the greater dispersion of material at the upper bend, greater wear was experienced at this point. Data were kept on the performance of a number of these hoses, the following being a representative and average test:

Date placed in service	September 22.
Date blown out	November 12.
Yards placed through hose	2 181
Hose[7] cost per yard	$0.0465

The lower bends were approximately 75% longer, although the angle of bend was the same.

A large number of 30° wrought-iron pipe bends were used as substitutes for the hose with greatly varying results. With extra heavy pipe, 5 ft. long, evenly bent its entire length, the wear of six pairs of bends gave the following average:

Average number of yards, upper bend	425
Average number of yards, lower bend	721
Cost per bend complete with flanges	$21.00
Cost per yard, upper bend	$0.0505

Two methods of patching worn-out bends were developed. The first consisted of welding heavy patches of steel over the worn spots. These patches when placed in service gave approximately the same results as new bends, but the practice was discontinued because after an indeterminate amount of wear the heavy patches blew off with grave danger to the crew working about them.

The second method was an outgrowth of the first and consisted of clamping several thicknesses of used placing-hose over the worn spot with heavy steel straps. This worked very successfully and as many as 750 cu. yd. were placed through the bends before the patches wore out. This method is effective in making a quick repair while pouring without removing the bend.

Patched bends, however, are the potential cause of plugs. In spite of a slight difference in cost in favor of the pipe bends (when patches are used), the fact that plugged discharge lines are much more infrequent with hose bends is the weightiest argument in their favor. Plugs in 99 cases out of 100 occur at bends. With the pipe all that can be done is to disassemble the line and clean it out. With the hose a few well-directed blows with a wooden maul at the point of obstruction will almost invariably release the plug. (This is not so effective with the stiff wire-wound hose). Another advantage of the flexible hose is the fact that thin spots can be located by feeling the hose, whereas there is no way of telling how badly worn an iron pipe is without getting at the inside.

Flanges.—Instead of using standard six or eight-hole companion flanges a slotted four-hole flange with attached swinging bolts was developed. These materially cut the time consumed in removing short lengths of pipe as the concrete advanced toward the forward bulkhead and prevented the loss of bolts and nuts.

[7] Hose given one-quarter turn after every 125 yd.

Care of Machine.—Although the number of moving parts in the machine are few these few need constant care to prevent undue wear. The standard worm furnished with a new machine is too light for constant service, the fin being made of ¼-in. boiler-plate spiraled around the hub and riveted to it through brackets. At the start of the Cascade Tunnel work these worms were a source of constant and annoying delays. (It is a 4 to 6-hour job to replace a worm.) In several cases the fin was torn completely off the hub by thin pieces of rock that jammed between the lower edge of the fin and the cylinder liner. This difficulty was remedied by the use of a specially designed worm cast of special abrasion-resisting steel with the fin ½ in. thick at the edge and ¾ in. thick at the junction with the hub. None of these cast worms broke and only two replacements were required in any machine because of wear. After the edge of the fin wears back more than an inch, the delivery of material to the outlet recess is so slow as to affect progress materially.

No trouble is usually had with the bearing (closed end) at the forward end of the cylinder, but frequent inspections are required to avoid trouble with the bearing at the motor end. The packing in the gland is quickly cut out by the air pressure forcing thin grout along the shaft. Lubrication must be attended to hourly by turning up the grease cup on the bearing. If this bearing and the corresponding bearing of the transmission case are not looked after the grout will work along the shaft and enter the transmission case. Two instances of the transmission case being almost filled by grout mixed with oil came under the writer's observation. In one case the grout had set so hard that the gear train was frozen in it. Due to the dust which is the usual accompaniment of concrete operations the motor requires more than ordinary care. When replacing the worm it is a very good idea to examine the liner plate and if it is materially worn to replace it at the same time.

Air Supply.—The results of experiments to determine the air requirements of this pneumatic concrete placer may be tabulated, as follows:

Number of batches poured	56
Size of batch, in cubic yards	0.565
Total number of cubic yards poured	31.64
Number of charges of gun	98
Amount per charge, in cubic yards	0.32
Quantity of air used, in cubic yards per minute	1 600
Length of time that air is used for each charge of the gun, average in seconds	14
Amount of air used per charge of gun, in cubic feet	373
Air in cubic feet per cubic yard of concrete placed	1 134
Pressure, in pounds per square inch	105
Length of 6-in. placing hose, in feet	12
Length of 6-in. placing pipe, in feet	16
Combined length, in feet	28

With an air pressure of 105 lb. as used in this test, consumption would probably be a maximum. Experience at the Cascade Tunnel indicated that a compressor of much smaller capacity can be utilized if a good-sized air receiver is mounted near the gun. Since the demand for air occurs at regular intervals it is practicable to build up a supply while the air-control valves are closed. High pressure is not essential to the successful operation of the

machine; in fact, with adequate reservoir capacity the best results are had at a pressure of from 70 to 80 lb., and the apparatus will function very well with pressure as low as 60 lb.

Safety Precautions.—It should be remembered that the apparatus is periodically under rather high internal pressures and should be treated with the same respect as a steam boiler of equivalent pressures. Connections should be examined and tightened periodically (they have a habit of loosening from vibration), and flange threads should be inspected for wear. This is particularly important over the forms because the puddlers usually work in constricted quarters and have little chance to escape from bursting pipes.

When hose is used in the discharge line in proximity to wires carrying electric current the iron pipe near the discharge end of the line should be well grounded, thus avoiding the possibility of electrocuting workmen in contact with the discharge line above the forms.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

MEASURING IRRIGATION DELIVERIES IN THE PANJAB

By E. S. Lindley,[1] M. Am. Soc. C. E.

Synopsis

The paper describes the conditions of Indian canals, as affecting the problem of distributing the supplies received in the manner desired. It describes the devices available for effecting such distribution with a minimum of manual control, mentions their principal qualities, and refers to the records of investigations directed to their improvement. An outline is given of the theories on which to base design in using such devices to form a complete scheme of automatic distribution; and, finally, their practical application is discussed, in so far as they are found possible under present limitations of knowledge and invention.

The method and means of measuring and apportioning irrigation deliveries, that are in course of development in India, and especially in the Panjab, are described. The writer, however, has not the local knowledge needed to judge whether, and how, these could be applied to American conditions. It has been the writer's aim to present essentials in such a form that discussion will be possible.

General Conditions

The irrigation canals of India are built, owned, and managed by the Government, which sells the water to the actual cultivators of the land served. The water is paid for at fixed acreage rates levied on the fields actually cropped on canal water in any season. The reason for this is that hitherto it has been impracticable to measure the water delivered to a holding, or even to a group of holdings, while field-to-field assessment was already in force for

Note.—The Special Committee on Irrigation Hydraulics has selected the subject of "Irrigation Deliveries" as one of ten for study and research. This paper was submitted to the Committee by the author, and the Committee has recommended its publication in *Proceedings* in order to elicit discussion of the subject. (See Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1929, Society Affairs, p. 97.)

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the rent or land revenue levied by the Government as the primary owner of all land.

Water rights are not fixed by volume, or by a rate of flow demandable at will. For each group of holdings a rate of flow is calculated from its area and other conditions. The canal system is designed to deliver these flows to all holdings when it is running full; when the supply is insufficient to fill the canal, the water is shared proportionately. For a variety of reasons, some local and some general, the simplest and best method of sharing has been found to be to keep each subsidiary channel running full or closed in short turns, for the correct proportion of the entire time.

The channels of a canal system fall into three classes: (1) Watercourses (farmers' ditches) serving the holdings that can be grouped together on one such channel, into not too large a unit; (2) distributaries and minors, feeding such watercourses; and (3) main canals and branches, feeding distributaries, but not watercourses. The distinction between Class (1) and Classes (2) and (3), is that the former is managed and maintained by the farmers, subject to the duty to keep the watercourses in good condition. The distinction between the second and third classes, both being managed and maintained by the Government, is that Class (2) channels are run only full to feed their watercourses correctly and automatically; while those of Class (3) may be run at any discharge, with the off-taking discharges controlled according to plan.

The supplies of the more important canals of the Panjab are secured by weirs; thus, they can be run up to their capacity as long as so much water is available in the river; but there is no storage for supplies in excess of momentary requirements. The canal systems vary in length up to 200 miles, and water takes as long as five days from the head to reach the last fields. (See Fig. 1.) Some canals have no weirs, so that they have to utilize the supply entering them from seasonal river levels. Many canals outside the Panjab are fed from storage of the monsoon flow of rivers which are almost dry for part of the year. This paper deals principally with the conditions of a large perennial canal of the Panjab. Fig. 1 shows the degree to which the Panjab plains are covered by canal irrigation. The areas commanded by "perennial" (weir-fed) canals are shown hatched; those commanded by "inundation" canals (fed only by natural rises of river level) are dotted.

The heads of main canals, branches, and main distributaries necessarily contain mechanism for the manual control of the supplies passing. The criterion in control has hitherto been satisfaction of the demand of those channels that were intended to receive supplies at any time. The correctness of these demands has been checked by periodical discharge observations at the heads of channels, and at the boundaries of the management districts. This method of check has become increasingly unsatisfactory as development of the irrigated area has caught up with the supply available. Deficiencies are due to the unavoidable errors of observation, considerable "personal factors", and accumulation or scour of silt which changes the rating curve. Progress is now being made with the provision of meters to eliminate discharge observa-

tions. The task of calibrating existing works by individuals, or of designing standard calibrated works, is of old standing; the present tendency is to provide "semi-modular weirs or flumes" on the lines described subsequently and often by only a slight modification of existing works.

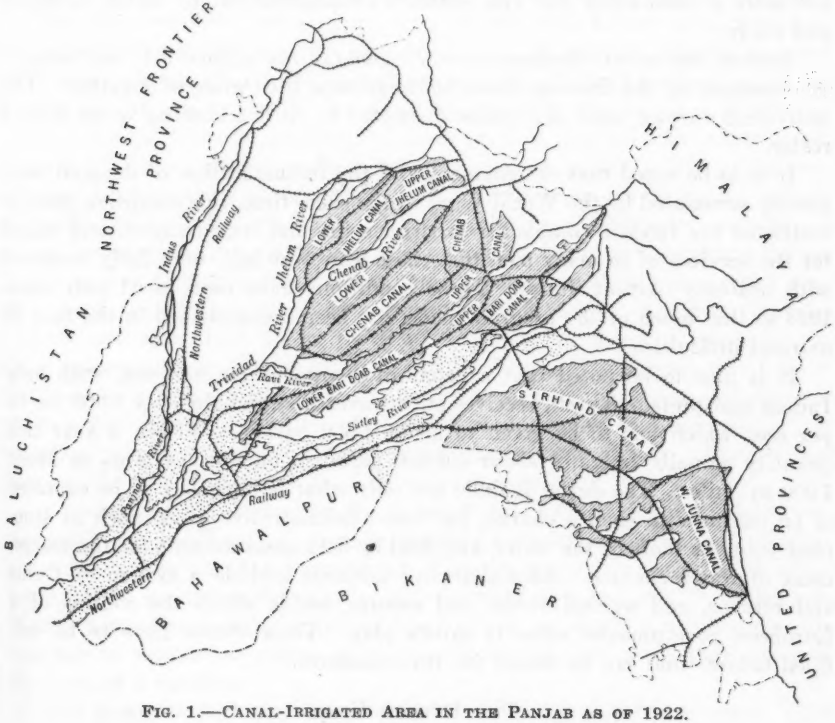


FIG. 1.—CANAL-IRRIGATED AREA IN THE PANJAB AS OF 1922.

The heads of minor distributaries were also controlled by hand, but it has now become fairly general to effect distribution at such points automatically, by pairs or groups of semi-modular flumes.

The heads of watercourses, or "outlets", were originally mere open cuts in the distributary bank. Several decades ago pipes and culverts in the bank were substituted for these cuts; and it is instructive to note that at the time this substitution met as much opposition as is now offered by some to more modern ideas. The discharge of such outlets was dependent on the water level, both on the distributary and the watercourse side. While the former was relatively constant, especially under a policy of running a full supply or nothing, the latter was liable to considerable variation, according to the level of the field being irrigated, and to the state of neglect or clearance of the watercourse, depending on keenness of demand. The heads available did not make it generally possible to eliminate the effect of water level on the watercourse side by setting the outlets with a free fall. The first suggestion for a device with which hydraulic principles might be used to provide a virtual free fall

was made about 1908, and, at present, automatic distribution is being rapidly improved by extending and improving the use of such devices.

The question is no longer "whether", but "how". The "how" is still in the stage of trial and intense discussion, both as to general policy and details; but there is sufficiently full and concrete accomplishment by which to report and study.

Beyond the outlet, discharges are, in general, not subdivided; the supplies are managed by the farmers whose holdings have been grouped together. The individual farmer takes the entire discharge in turn according to an agreed roster.

It is to be noted that development and the intensification of demand were greatly stimulated by the World War; at the same time, this condition gravely restricted the funds available for experimenting and improvement, and called for the services of so many men that those who were left, were fully occupied with ordinary current duties. These conditions have been eased only since 1926 so that much of the progress made, has been accomplished in the face of unusual difficulties.

It is also to be noted that a canal in charge of an engineer, with only Indian assistants and staff, receives on the average approximately 1 500 cu. ft. per sec., which has to be given to about 1 500 outlets. Within a year this quantity actually irrigates about 400 000 acres, in an area as large as about 1 000 sq. miles. The duties include not only what is obviously to be expected of an engineer in such a charge, but also administrative work, such as stopping waste or theft of the water, and field-by-field measurement for the assessment of water charges. Administrative methods include a system of forms and returns, and written orders and reports, beside which the routine of a European governmental office is child's play. These duties have to be fulfilled before time can be found for investigations.

The Devices Used

The term, "module", as used in this paper, is defined as a device designed to deliver a constant discharge, independent of fluctuations of water level or pressure on either the supply or the delivery side.[2] The first to develop any of the devices that have proved of practical importance was the late Mr. R. G. Kennedy, of the Panjab Irrigation Department, whose investigation of channels that just carried their silt charges without scouring, is widely known. He first devised three types of "modules",[3] for big, medium, and small discharges, covering the entire range of requirements. Of these, the medium and small types involved regulation by balanced cylinders suspended from a weigh-beam, and have not proved suitable for practical work. The largest consisted of a gate swinging on a horizontal axle, with a counterweight linked to an extension of its upper end and running on a guide-rail laid to a curve calculated and confirmed by experiment; and the basin under the bottom of the gate was also formed to a longitudinal curve. Any given difference of

[2] Panjab Eng. Congress (1923), p. 45.

[3] Panjab Irrigation Branch *Paper No. 12* (blue), of 1906, "On the Distribution of Water for Irrigation by Measurement".

up-stream and down-stream water levels brought the gate to some position predetermined by the counterweight and the curve of its guide-rail, and the basin graduated the aperture at each position so that the desired discharge would pass. The device was made in several sizes, and each could be set to furnish the discharge desired, by giving the correct amount of counterweight.

This "gate module" was used for a few distributary heads, which then had to be equipped with modules for special reasons. It seems to have functioned quite satisfactorily, but when it had removed the difficulties that led to its installation, it was gradually forgotten and ignored. At one site, conditions (not concerned with the module) led to so much silting that the water levels rose above those that the module was intended to control. The device was removed because those concerned did not know how to put it right. Efforts are now (1931) being concentrated on regulating discharge at outlets; at minor heads, needs are more simply and probably better met by flume furcations; at distributary heads, similar flumes to meter manually-controlled supplies seem to satisfy present requirements. This device, therefore, may be said to have been "born before its time", and thus far seems to have met the usual consequences. Fig. 2 shows a pair of flumes dividing the tail flow of a distributary between two form ditches. The one in the foreground has not quite a free fall, but the flow at the tail of the flume is unsubmerged. A standing wave is formed a little below the narrow throat. Distribution between the two is the same as if both had free falls.

A "gauge outlet", devised by Mr. Kennedy in 1908, is the first of the "semi-modules", which differ from modules proper in rendering discharge independent of water levels on the down-stream side, the natural variation with water levels on the up-stream side continuing in accordance with the hydraulic law. As the down-stream variation is the greater, and is largely controlled directly contrary to regulation policy, this class of device does more than one-half the work of a module.

This gauge outlet[4] may be described as a modified Venturi meter. The modification consists in providing free admission of air all around the throat, so that static pressure in the jet at that point cannot fall below zero. There is the usual recovery of kinetic energy in the expanding down-stream section, so that zero pressure is possible at the throat when that is lower than the delivery water level. It will be understood without lengthy explanation, that the discharge is independent of down-stream levels as long as the difference of head is sufficient to draw the throat pressure down to throat level; if it is sufficient to draw it lower, air bubbles will be attracted into the stream, and the excess head will be wasted. The discharge then depends only on the depth of submergence of the throat by the supply level. The outlet is not adjustable, but the desired discharge can be given by setting it with the necessary submergence, subject to the maximum allowed by the difference of head available.

The outlet has been widely used. It has brought about a considerable improvement in distribution, and blazed the way for most of the further work

[4] Panjab Irrigation Branch *Paper No. 12* (brown), "Results of Tests of Mr. Kennedy's Gauge Outlet"; together with addenda on its improvement, protection, and application.

that has been done on this subject in India. After it had shown that the needed improvements were possible, however, certain defects gradually became apparent.

From the beginning, the most serious defect has been that it is perfectly easy to stop the air supply to the throat, in order to increase the supply received. In the absence of any public opinion among the Indian peasantry, it is difficult to deal with this interference, and impossible to stop it. Some means of meeting this defect have been devised, but the outlet has fallen completely out of use in the Provinces with which the writer is in touch, and he has heard nothing of any actual use of these newer models.

Other defects are the natural result of the textbook theory that dictates the measuring of the head which causes velocity, down to the center of the orifice through which the water flows. To make it theoretically an orifice discharging freely into air, Mr. Kennedy provided an enlarged section in the throat in such a way that a supply of air surrounds the jet. He seems to have assumed that the top and bottom of the throat would be aerated simultaneously; and he claimed that the appearance of air bubbles in the discharge was a certain and simple indication that the device was discharging as designed. In unpublished tests the writer has found that, between the condition in which discharge was fully dependent on difference of water level with no air bubbles passing, and that in which it was fully independent of downstream level with air bubbling freely, there was an intermediate condition with discharge less than the theoretical, but with bubbles appearing.

A further result of following this "center of orifice" theory involved no practical defect, but is of interest. Mr. Kennedy calculated the coefficient of discharge from observed discharges and submersions measured from the center of the orifice as zero point. He was impressed by the fact that the values of the resulting coefficient were so low, and that the coefficient decreased with decreasing head. The writer plotted V^2 against water level, and found that if the zero point were taken at almost two-thirds of the height of the orifice, the coefficient was constant down to quite small submersions, and that it had a reasonable value for the bellmouth used. Another instance in which the writer was able to point out that the "center of orifice" theory had been misapplied, was the calibration of the sluices of the Assuan Dam.[5] The same error had also been found in investigating relatively short pipes discharging freely into air.[6]

Both these two last points would be avoided by making the orifice rectangular, or at least aerating only a flat top, as in the standing wave orifice semi-module mentioned subsequently in this paper.

A defect that is not yet generally felt, is that the discharge can only be adjusted by means of the depth of submersion, which should be fixed by other considerations to be discussed later. That defect is overcome by another form devised by Mr. Kennedy.[7] In this form the throat which constitutes the

[5] *Minutes of Proceedings*, Inst. C. E., 1921, Paper No. 4350, pp. 50 and 71.

[6] Ministry of Public Works, Egypt, Research Work, Delta Barrage, "The Discharge of Pipes", by A. Deane Butcher.

[7] "The Irrigmeter: An Irrigation Distributor and Meter, as Specially Adapted to American Methods of Distribution," by R. G. Kennedy, 1918, Colonist Press, Victoria, B. C., Canada.

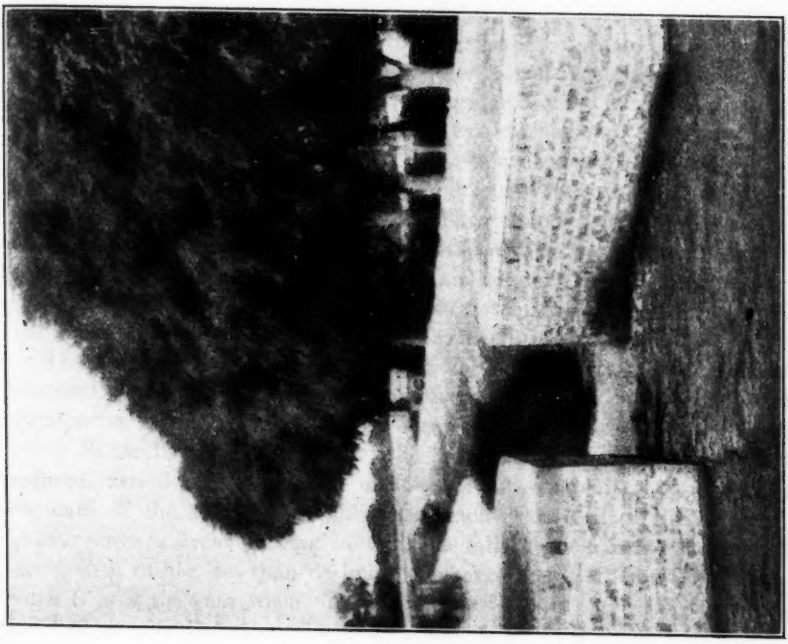


FIG. 3.—A STANDING WAVE FLUME.

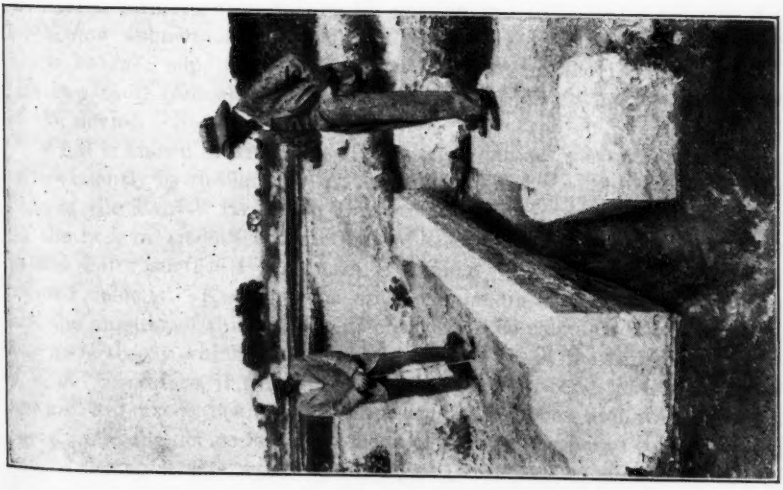


FIG. 2.—FLUMES DIVIDING THE TAIL FLOW OF A DISTRIBUTARY BETWEEN TWO FARM DITCHES.

measuring orifice, is set vertical, and a cone is suspended in it for the purpose of adjusting the area available for flow. This form has not been brought into practical use, within the writer's knowledge.

In about 1906, Mr. A. S. Gibb, formerly of the Panjab Irrigation Department, devised what is believed to be the only true module in which nothing moves but the water.[8] A flume of rectangular section follows a circular arc. The flow, therefore, must be part of a free vortex, and the surface of the designed discharge can be calculated. Vanes are set across the flume, leaving the designed discharge free, but skimming off any excess and turning it toward the center and causing turmoil. This module must be set unsubmerged by delivery levels; the supply water level must be high enough to give the designed velocity, and must not exceed this by more than the amount of head that the vanes provided can waste. It is not made adjustable for discharge, but is designed for discharges varying by small steps.

The use that has been made of this module is, as far as the writer is aware, practically confined to complete regulation of one long channel in which the distribution among watercourses gave great trouble with ordinary culvert outlets. Naturally, the cultivators who had their former excessive supplies reduced, raised objections and made constant complaints. Judged by the standard of the time (the supply that reached the tail of the channel), the measure was a great success in that the tail received rather more than its share, with rather less than designed discharge admitted at the head; and the same is still the case when, for a number of years, the local engineers have not had to give the channel special attention.

However, the fact as stated indicates that some outlets receive less than their share. The earlier forms of the module were found to be slightly incorrect, but have been replaced by a later form. A recent inspection showed that some farmers were apparently getting too little water because delivery levels now submerged the modules; and others were apparently getting too much because supply levels had now risen above the modular limit. The last two facts indicate slight disadvantages to weigh against the simplicity of the device.

What is known in America as the "Venturi flume" was originated earlier and independently by the late Dr. A. A. Stoddard and the late Mr. W. B. Harvey, both of the Panjab Irrigation Department. By introducing a smooth hump on the bed, or smooth contractions of the sides of a regular channel, a dip in the water surface is produced equivalent to the head converted into increased velocity. Knowing the areas of the up-stream and throat sections, and the amount of this dip, the discharge can be calculated; but by the standing wave theory which is now widely known, V^2, at the throat, cannot exceed $g \times d$. Therefore, if the contraction is sufficient, conditions from the throat upward are fixed; below it, a standing wave forms and wastes any excess energy, and the device is a semi-module with its discharge constantly dependent on the up-stream water level only.

[8] Panjab Irrigation Branch *Paper No. 13*, "Gibb's Module; Report on Its Working on the Shahkote Distributary."

In the Panjab, this device was first introduced to distribute supply among the outlets grouped at the tails of distributaries. These supplies are even now variable; therefore, the first need is to distribute them proportionately, in spite of differences between, and fluctuations of, delivery water levels. This need would not be met at all by the two-gauge Venturi flume, and the one-gauge standing-wave flume has been the one developed. Fig. 3 illustrates a standing wave flume used for automatic semi-modular division of supply between two small branches. The flume shown in the center is discharging about 30 cu. ft. per sec., with an up-stream depth of about 2.5 ft. With an afflux of less than 6 in., it is discharging as if it had a free fall. "Modularity" with small working heads was the first desideratum, and as the difference between ordinary and extreme efficiency meant only a difference of about 0.1 ft. in maximum permissible delivery level, even this demand was mild. Accuracy of discharge was not at first very important because the discharges dealt with had been subject to 100% variation and a reduction of this to, say, 20%, satisfied all concerned. Both have become much more important since the device has been used on a much larger scale. The *prima facie* simplicity of elementary theory is found to be complicated by points that this theory ignores; and much important work has been done to perfect the device.[9]

The simultaneous investigation of the device in America has concerned itself more with the two-gauge Venturi flume. Recently, advance seems to have led more to the one-gauge flume.[10] It seems to the writer that the ample resources available for American research would achieve even more valuable results, if a study were also made of what engineers in India have accomplished with very cramped resources.

When Messrs. Stoddard and Harvey originated their flume, they also devised an orifice gauge outlet based on the same principle. In this, a bellmouth leads to a rectangular orifice; instead of the closed expanding pipe of the outlet as developed by Mr. Kennedy, this continues as an open flume the bed and sides of which carry on those of the orifice. Under appropriate conditions, a standing wave forms in the flume, leaving the orifice free to discharge into air with a supported jet.

The outlet has the advantage of extreme simplicity; but under many conditions a relatively large working head is necessary for the standing wave to form. This led Messrs. Stoddard and Harvey to concentrate on an attempt to make smaller heads sufficient. These attempts produced some extremely ingenious ideas, but achieved no practical success. In spite of this limitation, the outlet[11] had a considerable field of utility in this simple form, and has been used successfully. A further considerable advantage is that the outlet is adjustable, by making the roof of the bellmouth and orifice slide up and

[9] Unpublished investigations of E. S. Crump, Panjab Irrig. Dept., Panjab Eng. Congress, 1927, "Report on Flume Experiments on the Sirhind Canal," by A. G. C. Fane; Bombay Public Works Dept., *Technical Paper No. 15*, "Notes on Standing Wave Flumes and Flume Meter Falls," by C. C. Inglis; and Panjab Eng. Congress, 1925, "Canal Falls and Their Use as Meters," by E. S. Lindley, M. Am. Soc. C. E.

[10] "The Improved Venturi Flume," by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 841.

[11] Panjab Irrigation *Nomograms*, Pl. 45*a*, and the pamphlet of the manufacturers.

down the sides. A special form was also constructed,[12] which diminished the working heads needed by using the outlet with shallow submergence, for proportional distribution of supplies. The relative advantages of that and other distributions are discussed in this paper.

After considerable further work by other engineers, the problem of saving head seems now to have been solved by Mr. W. Kirkpatrick, of the Bombay Public Works Department. He sets vanes across the flume of the outlet, with their lower edges on an incline rising from the orifice. These help to support the standing wave, and the orifice is free with a satisfactorily small working head.[13]

A number of devices have not been mentioned which, like Kennedy's two smaller modules, have never found application and seem unsuited to irrigation conditions in the field. The same applies to devices that are really only minor modifications of the originals described. The literature that has been quoted in references contains brief descriptions and references to any published, more complete descriptions.

Only one further group of devices calls for mention here. One is a module intended for outlets, which is now used to regulate a constant supply for sand filters. It is of the type in which a piston "floats" on the difference of head across a fixed orifice, and as it moves it adjusts a second orifice so as to waste more or less head and keep the former head constant. It has been tried in the Bombay Presidency and is understood to have proved unsuccessful because of the long streamers of weed that grow in the water and the colloidal mud which accumulates on the guide-rod for the floating piston. It was applied to the outlets of a small distributary in the Panjab and this installation has never been properly examined since, but on one occasion the writer had the opportunity to go over the channel and found distribution still satisfactory.

A second device is a module intended for large discharges. It is a Venturi meter provided with a butterfly valve a little down stream from the throat. The valve is actuated by reversible mechanism controlled by the Venturi head, set to be stationary at the designed head, to close with greater head, or open with smaller heads. A few were installed for trial in Bombay, but the writer has not had any certain report of their working.

The Venturi meter has been used for some very large discharges in Southern India and is believed to be satisfactory. In the Panjab its field is filled by the flumes described herein.

Methods of Using the Devices

At first, "moduling" was done for outlets individually; but methods of collective use have since been developed, so that the supply admitted at the head of a canal is automatically distributed to the outlets on it. At first thought, it might appear that if modules were fitted to the head of the channel and to all outlets, this result would be simply secured; but no positive module has yet been definitely shown to be so reliable and satisfactory as to justify

[12] Panjab Irrigation Branch *Paper No. 26*, 1922, "Moduling of Irrigation Channels," by E. S. Crump.

[13] Bombay Public Works Dept., *Technical Paper No. 12*, "Jamrao Semi-Modules," by W. Kirkpatrick.

the cost. Even if that were the case, fluctuations would still have to be faced, as, for example, when cultivators close their outlets for silt clearance without notice, or when channels have to be overloaded temporarily while a breach is repaired.

The ideal aim for a plan of "moduling" a distributary may lie anywhere between two extremes: One is to give each outlet and the distributary the full designed supply, and to close the distributary as required; the other is to give each outlet its correct share of the supply passing at the moment. The former is theoretically possible; but for practical reasons referred to previously, it is accepted by all as necessary to fit the tail group of outlets with flumes in order to distribute proportionately the supply reaching them. With the devices available, the latter ideal is theoretically possible only by means of a pair of flumes at each outlet, distributing the supply proportionately between the outlet and the distributary. As this would involve a small drop of distributary supply level at each outlet, it is not really practicable. What is practicable and practised is to assume that the supply level corresponding to a given discharge is constant, and to fit each outlet with a semi-module intended to take off the proper proportion of supply as long as this holds good. As the assumption in practice fails to a varying extent, this latter ideal is not fully possible; but before proceeding to more detailed discussion of the two extremes and of intermediate possibilities, it is necessary to state the theory of the subject that has developed.

Theory of Group Distribution

This theory is based on the idea of proportionality, which is most simply explained in algebraic form.

When giving each outlet a fixed proportion of the supply, if D is the designed supply of a distributary and if nD is the designed supply of the outlet, when $D \pm d$ is the actual supply in a distributary at any moment, then the outlet receives $n\ (D \pm d)$ gal. of water per hour. It is always necessary to provide that the outlet shall receive nD gal., when D gal. are available; but fluctuations can be otherwise dealt with, and if the outlet receives $n\ (D \pm rd)$ gal. of water per hour, the fraction, r, is by definition the "proportionality".

It has been found that the discharge of silt-formed channels in the Panjab varies almost exactly as the $\frac{5}{3}$ power of the depth.[14] If an outlet is a flume in which the discharge varies as the $\frac{3}{2}$ power of the submergence, set with its sill at one-tenth the designed supply depth above the distributary bed, with the width such as to give the correct discharge at the designed supply, the curves for proportional and actual discharge would be tangential; and since the two exponents, $\frac{5}{3}$ and $\frac{3}{2}$, are not very dissimilar, the two curves do not diverge enough to call for practical objection; that is, such a flume is a semi-module giving sensibly proportionate distribution as long as the rating curve of the distributary at the point does not change.

[14] Panjab Eng. Congress, 1924, *Paper 89*, Paragraph 5, *et seq.*

It has also been shown mathematically, that an orifice semi-module discharges proportionally when the submergence of its zero point is three-tenths the designed supply depth.[15] However, in this case the two exponents, $\frac{5}{3}$ and $\frac{1}{2}$, are dissimilar, involving opposite curvature.[16] They are only tangential at a point, and away from it they diverge rapidly. Such an orifice is, therefore, "proportional" to a much more limited degree; but that objection is partly overcome by the device developed by Mr. Crump.[17]

Expansion of this mathematical treatment results in the conclusion that, for a flume the submergence of which is the fraction, h, of the supply depth, the proportionality, r, equals $\frac{0.9}{h}$, and for an orifice, r equals $\frac{0.3}{h}$. These formulas give the measure of the degree to which the performance of a semi-module may be made to approach that of a module by setting it deeper.

If the outlets of a length of distributary are fitted with modules, all the variation greater or less than the designed supply reaches the tail, since none of the outlets receives more or less than before. On the other hand, if all the outlets received proportional supplies, the percentage variations at the head and at the tail would be equal. Between these extremes, the use of semi-modules set with proportionality of less than 1.0 magnifies the percentage variation of supply from point to point down the channel; and the mathematical treatment referred to gives the measure of this magnification. The mathematical assumptions must not be overlooked, namely, that the variations are small, and that the outlets are infinite in number and of infinitesimal discharge. Let,

r = the proportionality of the outlets used.
S = the ratio of supply at the head of the reach to that at the tail.
V = the ratio of percentage variation from the designed supply at the head, to the percentage variation resulting at the tail.

Then,

$$V = S^{r-1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Subject to the assumptions made, it is then possible to calculate and design a system with semi-modules for outlets and furcations of channels with a desired distribution not only of the designed supply, but also of variations from the designed supply.[18] For example, if it is necessary to be prepared for each distributary to admit, in an emergency, an excess of 2½% at the head, while up to 20% excess can be carried at the tails without danger, and it is desired to use orifice semi-modules set near bed level (proportionality = 0.3), it will suffice to fit flumes that are set for proportional distribution to tail outlets that receive one-twentieth of the head discharge.

[15] Panjab Eng. Congress, 1917, *Paper 36*, p. 137*f*.
[16] Panjab Eng. Congress, 1923, *Paper 80*, Fig. 1.
[17] Panjab Irrigation Branch, *Paper No. 26*, 1922, "Moduling of Irrigation Channels".
[18] Panjab Irrigation *Nomograms*, Plate 28*b*.

Practice of Group Distribution

If positive modules are used, it suffices to determine the proportion of tail outlets that must be fitted for proportional distribution, to form a sufficient safety valve. For want of a sufficiently satisfactory form of positive module, their use has nowhere yet become general. For some conditions such modules are very desirable, and even the nearest substitutes, are rather unsatisfactory. For example, in the Deccan, weed growth is apt to be so heavy that the supply level in a channel designed to run 3 ft. deep may rise by 3 ft. For other conditions, such modules are useless; as, for example, in inundation canals the supplies of which are not secured by river weirs, but depend on the natural rise and fall of the river. In such canals the best use possible must be made of the widely fluctuating supplies, and, therefore, they must have rather flexible distribution.

Proportional distribution has most adherents in the Panjab at the present time. This is probably in large part because designing is very simple and progress always takes the simpler steps first. It is found satisfactory that these measures effect great improvement on distribution by uncontrolled culvert outlets, and the limitations involved in the assumptions made to simplify theory are not realized. For example, the fluctuations of supply level due to silting cause departures from designed distribution which are greater than need be incurred. It is very easy, although illegal, for cultivators to steal quite a quantity of water by obstructing the distributary below their outlets.

Theoretical ideals being thus unrealizable, an intermediate policy is advocated as the best practicable course. This involves distribution, as nearly constant as possible, to the maximum number of outlets. It is effected by fitting the maximum number of outlets with orifice semi-modules set as deep as possible, confining proportional distribution to tail clusters, and using positive modules only where fluctuations of supply level are too great to allow of compromise.

This policy is naturally the subject of controversy in opposite directions with the advocates of the two ideal policies quoted. Positive moduling is likely to win for some conditions if and when types of modules are developed which compete with semi-modules in certain essentials. They must be cheap; simple and robust of construction and action; sufficiently accurate in discharge; susceptible of adjustment by the engineer, but not of undetected adjustment by the cultivator; they must work with water containing weeds and mud, and must not be deranged by flotsam and jetsam; they should require only a small minimum working head, and should have such range that they are easy to install and will cover actual fluctuations of supply level. It is also desirable that they show the discharge they are set for, and indicate (like the bubbling of a Kennedy outlet) when they are working. When such modules are produced, the field they can best cover can only be found in actual practice; for example, on a Panjab perennial canal the present tail clusters may still prove to be sufficient as safety valves. On the other hand, it may be necessary either to fit a few more outlets for proportional distribution, or retain a larger number with sub-proportional distribution.

For other conditions the final solution is likely to remain that of maintaining nearer proportional distribution, in spite of the realization that the practicable version of the theoretical ideal involves disadvantages to be avoided as far as possible. It is argued that, even on a Panjab perennial canal, fluctuations of supply are such that any approach to rigid distribution endangers the tails; where relatively rigid distribution has been used, this has been disproved for present conditions; and present conditions include much more variation of the supplies received than obtain as metering and moduling are extended. There is also a measure for more effective use of the tail safety valves to be implied.

If a distributary furcates into two minors, one large and the other small, both fitted for sub-proportional distribution, but with equal proportional tail clusters; and if the furcation into minors is also fitted with flumes for proportional distribution, the tail of the larger minor will experience much larger fluctuations than that of the smaller. A method has been shown[19] in which the proportionality of distribution between minors is adjusted so as to give equal fluctuations at both tails, enabling each safety valve to be fully used.

[19] Panjab Eng. Congress, 1923, p. 67, Paragraph 84 *et seq.*; and Panjab Irrigation *Nomograms*, Pl. 28*b*.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

SURVEYS FOR SWIFT RIVER RESERVOIR OF THE BOSTON METROPOLITAN WATER SUPPLY[1]

BY N. LEROY HAMMOND,[2] ESQ.

SYNOPSIS

It is the purpose of this paper to outline, briefly, the methods pursued in the work of real estate and topographic surveys preparatory to the construction of the Swift River Reservoir.

The methods adopted are similar to those used on other large projects, with modifications to meet local conditions of valuation and time. On this work, the aerial survey has been used as a guide to the ground survey rather than in the final preparation of maps.

INTRODUCTION

The water-shed of the new Swift River Reservoir for the Metropolitan Water District of Boston, Mass., embraces an area of about 186 sq. miles. Of this, about 88 sq. miles will be required for reservoir purposes, such an area including land within and extending approximately ½ mile from the proposed reservoir. The area of operation involves practically all of the Towns of Enfield, Greenwich, and Prescott, Mass., and parts of at least seven other towns.

The Swift River is formed by three main branches (see Fig. 1), the East, Middle, and West Branches, so-called. The water-shed of the river is characterized by a series of high ridges running generally parallel in a northerly and southerly direction. The middle and upper part of the main valley is occupied by a number of rugged, mountain-like rocky hills which rise 300 to 500 ft. above the valley floor. The valley is sparsely settled, having had a population in 1920 of about 20 persons per sq. mile, who lived mainly in the small villages in the valleys of the principal streams. Small areas in the valley, however, were devoted to agricultural use, the remainder consisting for the most part of woodland varying from scrub growth to mature timber.

Work of surveying the large area involved was started in the late fall of 1926. The surveys involve two major divisions—that of detailed topography within the basin of the proposed reservoir, extending at least 10 ft. in eleva-

[1] Presented at the meeting of the Surveying and Mapping Division, Boston, Mass., October 10, 1929.

[2] Div. Engr., Metropolitan Dist. Water Supply Comm., Enfield, Mass

tion above any probable flow line, and that of real estate involving the determination of all holdings within the reservoir requirements. Miscellaneous studies have been carried on coincidentally with the topographic and real estate surveys.

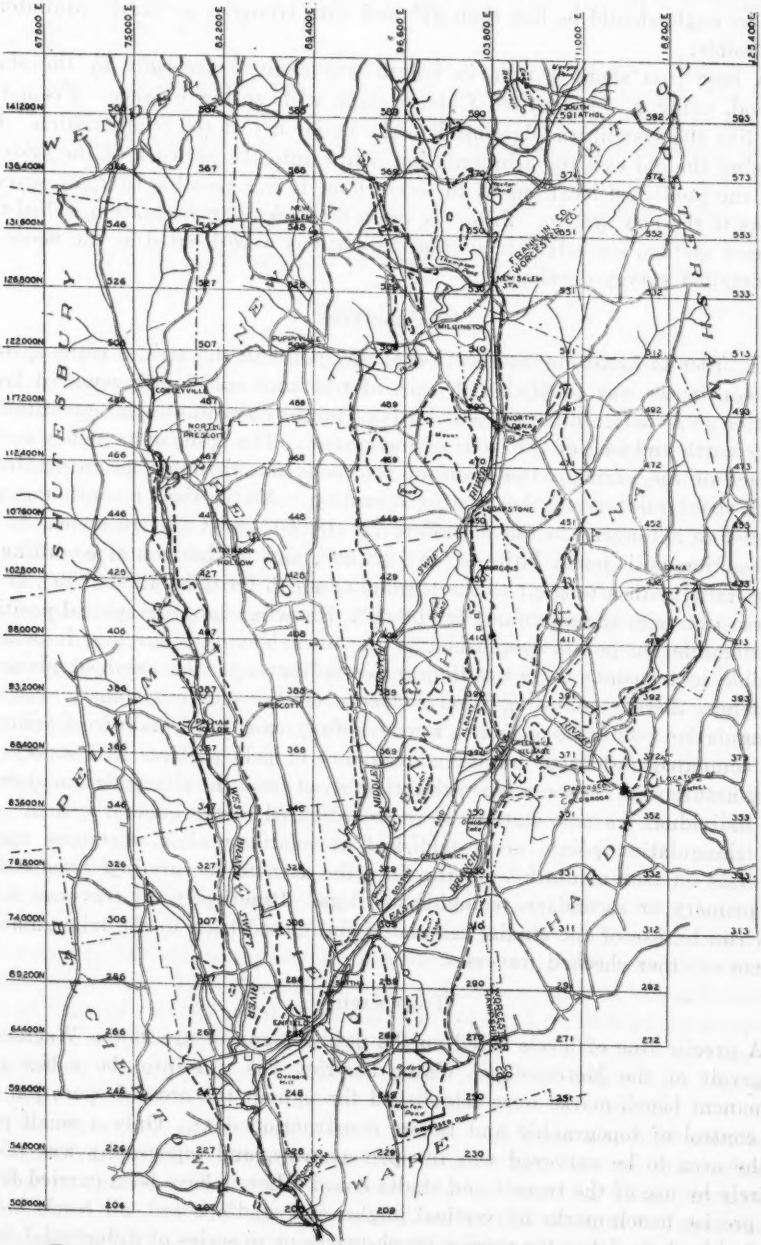
Triangulation

Control for all survey work has been obtained by a net of triangulation. The general terrain of the Swift River Valley was found to be ideal for a good system of quadrilateral network, except that considerable difficulty was experienced in the setting of stations so that they would be visible longitudinally along the axes of the valleys from quadrilateral to quadrilateral. Due to the fact that the hillsides are more or less uniform as to longitudinal slope, and, because of the occurrence of spasmodic growth of heavy timber, it was somewhat difficult to obtain open lines of visibility. Considerable care was exercised in the placement of stations in order that damage claims from the cutting of timber might be eliminated as far as practicable.

It was found that a considerable number of triangulation stations had been set in the valley or its environs by the former Harbor and Land Commission of the Commonwealth of Massachusetts and other governmental agencies. A few of these stations were in the secondary net used as a control of the Massachusetts Topographic Survey. In so far as it was practicable in developing a network of well-conditioned triangles, these stations were occupied and used in the general system adopted. The geographical positions of these early triangulation stations were published in 1912, by the Harbor and Land Commission in the atlas of the boundaries of the various towns of Massachusetts, and the citation of latitude and longitude lends an opportunity for a check on a system which could include some of the older triangulation points.

It is the thought of the writer that few engineers appreciate the quantity of accurate information and data available within relatively small areas for a check and tie-up of land surveys of all descriptions. Throughout the Commonwealth of Massachusetts there are many triangulation points of known geographical location, together with the positions of all town corners, of which careful descriptions supplemented by an actual survey sketch are given. A more general use of the data at hand would do much to correlate the many surveys which are constantly being made.

In the surveys of the Swift River Valley, the general network consisted of two chains of quadrilaterals covering two of the major valleys extending generally parallel in a northerly and southerly direction. At their upper and lower ends, the quadrilaterals were brought together in order that a proper check might be available for the entire system. In a few cases, polygons with an interior station were included in the system as well as a few simple triangles to make control accessible to certain districts. Common tripod signals, about 25 ft. high, were permanently erected on each of the stations of the system. The signals were constructed so as to permit setting the transit directly under the signal mast. Angles were measured by an engineer's transit graduated to minutes. Six repetitions of each angle were made and the error


67800 E
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78800 N
74000 N
69200 N
64400 N
59600 N
54800 N
50000 N
WENDELL
ALLEN
SHUTESBURY
NEW SALEM
ATHOL
ORANGE
NORTHFIELD
BERNARDSTON
DEERFIELD
GREENFIELD
WARWICK
MILLERS FALLS
MONTAGUE
LEVERETT
SUNDERLAND
WHATELY
HATFIELD
HADLEY
AMHERST
SOUTH HADLEY
GRANBY
SOUTH HADLEY FALLS
CHICOPEE
HOLYOKE
SOUTH HOLYOKE
WILBRAHAM
LUDLOW
SPRINGFIELD
EAST LONGMEADOW
LONGMEADOW
AGAWAM
WEST SPRINGFIELD
MONSON
PALMER
WARE
BRIMFIELD
HOLLAND
WALES
STAFFORD
SOUTHBRIDGE
WEST BROOKFIELD
BROOKFIELD
NORTH BROOKFIELD
EAST BROOKFIELD
BRIMFIELD
SPENCER
LEICESTER
WORCESTER
CHARLTON
OXFORD
DUDLEY
WEBSTER
DOUGLAS
GRAFTON
MILLBURY
SUTTON
NORTHBRIDGE
UXBRIDGE
BLACKSTONE
MILLVILLE
NORTHBRIDGE
WHITINSVILLE
NORTH GRAFTON
QUINSIGAMOND
LAKE QUINSIGAMOND
LAKE SIESICUT
LAKE SINGLETARY
LAKE LASHAWAY
LAKE MANCHAUG
LAKE WACHUSETT
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of closure of the figures of the network was generally kept considerably less than 8 sec. The triangulation network was laid out with the requirement that no angle should be less than 27° and with triangles as nearly equilateral as possible.

A base line about 1 mile in length was chained precisely by the slope method, using a standardized Chicago tape with spring balance. From the base line the system was developed to the major net of the triangulation. By utilizing the old existing triangulation points into the network of the system, with the published locations of these stations, it was possible to apply several checks to the new system. From the main network of triangulation, called the primary system, secondary triangulation stations were located as the needs of the detailed survey developed.

Co-Ordinates

In order to facilitate the location of any point in the area, a plane system of co-ordinates was adopted with axes of reference sufficiently removed from the area so as to eliminate the possibility of minus co-ordinates, all co-ordinates being north and east of the locus of the system. The point of tangency to the surface of the earth of the plane of reference was assumed as an arbitrary point about midway of the area of operation. All vertical co-ordinates are parallel to a tangent to the meridian, 72° 18′ 00″, west of Greenwich, at its intersection with north latitude, 42° 22′ 30″, and all horizontal co-ordinates are parallel with a tangent to the parallel of north latitude, 42° 22′ 30″, at its intersection with the meridian, 72° 18′ 00″. Knowing the geographical position of triangulation points established in the town boundary surveys, it became possible to translate these positions to co-ordinates in the assumed plane of reference, thereby affording several checks on the entire system.[3] For all triangulation points, co-ordinates were carefully calculated and direct azimuth was computed and listed for the convenience of field parties. All surveys of every nature were run on a true azimuth derived from the triangulation system, and individual traverse stations were co-ordinated to the general system. As the triangulation points are established at relatively short distances apart, traverses on true azimuth were run into the area to be surveyed from either the primary or secondary triangulation legs. Practically all traverses have been run by use of the stadia, calculated and closed back to the triangulation system or other checked traverses.

Topography

A precise line of levels based on the datum plane in use at the Wachusett Reservoir of the Metropolitan Water District was run into the valley and permanent bench-marks were established throughout the area of operation for the control of topographic and future construction data. Only a small part of the area to be surveyed was in open country, and topography was taken entirely by use of the transit and stadia board. Levels have been carried from the precise bench-marks by vertical angles and stadia board, all levels being checked back to either the precise bench-marks or to series of differential lines

[3] See "Geodesy," by George L. Hosmer, M. Am. Soc. C. E., 1919, p. 175.

run by use of the dumpy level. The topographic survey covers about 28 000 acres and sufficient detail is taken for the plotting of contours at 5-ft. intervals, at a scale of 1 in. = 200 ft.

In view of the fact that real estate survey requirements covered an area of great extent, involving the legal status of a vast number of parcels, the topographic surveys were carried on independent of real estate, except that the traverses established have been used for both the topographic and real estate work. However, the topographic survey included the location of all possible lines of real estate occupation to facilitate the actual real estate survey. The customary features of topographic information have been located, such as highways, watercourses, buildings, and cultural development.

Real Estate

Under its Legislative Enabling Act, the Commission may purchase land or acquire it by right of eminent domain. To date, (1931) the latter privilege has not been exercised and property has been acquired by title in fee. It is probable that ultimately a general taking line will be laid down accurately and all property within the area described thereby will be acquired. A considerable number of years elapsed in the preparation and final enactment of the Legislative Act creating the future Swift River Reservoir and most property owners within the area affected were cognizant of the need of their holdings, resulting in the early listing of much of the property for sale. Upon the listing of property its location was approximately determined by the field office, in order that need for its purchase might be determined. The property was then appraised by agents of the Commission, and steps were taken for its purchase.

Prior to the actual transfer of ownership, the title of the property was examined by the Legal Department, and if metes and bounds or acreage appeared obscure or inaccurate, surveys were requested of the Engineering Office. The largest percentage of the holdings in the valley was in either farm or woodland, more or less undeveloped, and it soon became evident that, in most cases, the survey of boundary lines and corners could be made by the stadia method to meet most requirements satisfactorily. It should be borne in mind that the major part of all surveys lies within a probable general taking line, and individual acquisitions will ultimately border other acquisitions, so that considerable economy in field work may be exercised by the proper use of the stadia method of survey. In the actual surveys, a rigid requirement was enforced in that at least two independent locations were required of each property corner where such corners are located as side shots from a main traverse and are not included in the traverse calculations. Such a method serves as a check in plotting.

Plots were made to a scale of 1 in. = 200 ft. All traverses are checked back to themselves or other checked traverses, lines being run on a true azimuth derived in the beginning from the triangulation net. As often happens in undeveloped country, such as that encountered on this work, it has been found that exact property lines are more or less obscure even to the knowledge of the owner and it becomes necessary to examine, carefully, deed and title of abut-

ting owners in order that all property in the vicinity of the surveyed parcel might be satisfied.

Title examination by the Legal Department was not confined to properties under immediate option for acquisition. It embraced all property which might be required. A systematic search of all titles in one of the towns was inaugurated and practically completed within one year. Four copy sheets of title abstracts were reduced by photostatic process to one letter-size sheet and given to the Engineering Office for its use. Thus, the Office had the advantage of being able to keep up to date with the title work and considerable economy resulted in the ability to survey considerable tracts of land in a systematic manner, with the result that to date (1931) about 58 000 acres of real estate have been surveyed in anticipation of request for such survey, or in the ultimate taking of all property.

Together with the preparation of maps for the actual acquisition of real estate by deed, an analysis of the property was made by the Engineering Office; that is, a review of all lines was written, with its relation to the deed held by the owner, as well as to the deed held by the abutting owner. The writer has found that assistants acquire a much broader outlook on real estate survey requirements by being obliged actually to write out their analyses for review by the Legal Department. A suggested description for possible deed conveyance also accompanied the map of the property when it was returned. This requirement has the tendency to make certain that necessary bounds, lines of occupation, distances, etc., appear properly on the map before it is passed to the Legal Department. The requirement of map analysis of deeds, and suggested description appeared at first to be somewhat burdensome to an engineering force, but as such analyses would be required mentally in the process of a survey, the actual writing of the story soon became a comparatively simple matter. It was found extremely valuable not only in the training of the personnel to a proper real estate survey, but also in the placing on record of the reason for each bound, line of occupation, or map line, embodied in the final map.

Aerial Surveys

Fortunately, prior to any actual work of intensive survey in the Swift River Valley, a contract had been made for an aerial survey of that part of the valley covered by the proposed future operation. The contract called for the photographing of about 100 sq. miles of area and the delivery of so-called contact prints. Photography from the aircraft was at an elevation of about 12 000 ft. above sea level, resulting in contact prints of about 7 by 9 in., with an average scale of approximately 1 in. = 1 100 ft. The contract also provided for enlarged prints to a scale of 1 in. = 400 ft. Because of the natural topography of the Swift River Valley with its series of high ridges, the individual contact prints were of areas with elevations varying in many cases as much as 500 ft., or more. With the resultant camera distortion, it was found extremely difficult to determine a suitable control scale for reproduction to a scale of 1 in. = 400 ft.

A variety of scales existed between various points of the same print, due to the considerable differences in ground elevation with resultant photographic

distortion. It was decided that, except by the utilization of considerable ground survey, the enlarged prints were of little comparative value; therefore, few contact prints were enlarged. In fairness to this method of survey, it may be stated that by considerable study for the proper placing of ground control, ratios could have been deduced so that a reasonably accurate mosaic could have been developed. For the purpose to be served it appeared that the contact prints in themselves were sufficiently clear and capable of being pantographed as the ground survey developed a control for the needs of the work. The contact prints were in more or less constant use from the time of their receipt and were extremely valuable in many ways.

Value of Contact Prints.—An inspection of aerial contact prints indicates, by lines of occupation or by variation in timber growth, the possibility of proper location in the field. In the cases of three of the towns affected by the Swift River Reservoir such lines were pantographed to a skeleton map of the town. With these data on the map a perambulation of the town was made in company with its assessors to determine the approximate ownership of lands within its limits. Lines which had no significance as property lines were crossed out in the field and a somewhat crude map of property ownership within the town was developed. Such maps have been of considerable advantage to title examiners in the tracing of ownership titles. The maps were executed simply as an aid to an accurate ground survey of the area.

Considerable use was made of the contact prints in the location of difficult lines and corners. Many times a difference of a few years' growth of timber would not be apparent on the ground, but the line of demarcation was shown on the aerial photograph and data were taken from the contact print indicating azimuth and distance for a field location of the desired lines or corners. The contact prints were useful during the process of checking the mapping of topographic detail, as any error in field work or of interpretation of notes was quite evident.

Cemeteries

Within the area needed for reservoir purposes were several cemeteries which required removal outside the reservoir limits. Each cemetery was surveyed, including a location of all monuments and markers, together with the exact location of the indication of all burials. Copies were made of inscriptions on all stones and so far as possible data were gathered as to actual lot ownership. A census of burials in the cemeteries indicated the existence of more than 5 000 interments.

A card index system was developed covering the data of each interment, together with information concerning lot ownership. In many cases the latter data are more or less obscure due to the absence of old town records; but as actual removals are made, public interest will make known much of the missing history of actual interments and of interested parties. To date (1931), four hundred removals have been made to lots outside the reservoir area, owned by parties at interest. No general work of removal has as yet been undertaken. A new cemetery is to be established by the Commission outside the reservoir

area for the re-interment of remains to be moved. Such a cemetery has not yet been located.

Mapping

All record plots of topography and real estate were made to a scale of 1 in. = 200 ft. The entire area was laid out to a network of standard sheets with definite assignments of sheet numbers to areas included within set co-ordinates. The standard sheets have an inside border dimension of 24 by 36 in., with no overlapping of data to adjoining sheets. Original plots of topographical and real estate data were made on the same sheet, consisting of white paper mounted on cloth, standard titles with graphical scale having been printed on the sheets before delivery to the field office. The sheets are oriented with their margins parallel to the co-ordinate lines previously described. Convenient working co-ordinates were carefully laid down on the sheets and as triangulation and traverse stations were calculated with their proper co-ordinate positions, they may be easily plotted. Detailed data were plotted by means of a paper protracter with an arm, or by a drafting machine. As all data were taken on the true azimuth which corresponds to the vertical co-ordinate lines, the matter of plotting was extremely simple.

Two sets of tracings were developed from the record maps, one showing topographic features and the other, real estate holdings. These tracings were of the same size as the record map with no overlap of data, one to the other. Standard titles with graphical scale were printed on cut tracing cloth before use by the field office, thus making for uniformity in the finished result.

With the early assignment of sheet numbers to areas included within definite co-ordinate limits, it became easily possible to control for reference, the location of any data taken. Each triangulation station was given a number which corresponded to that of the record sheet on which it would occur, and all data taken in the field were correlated to their proper sheet numbers. Loose-leaf note-books were used and the data taken were grouped together to form books that had numbers similar to those of the record sheet on which the information might be found. This system simplified the matter of keeping in order the information secured. If a field party was assigned to a definite task, it could (by locating the sheet number of the area in which it was interested) immediately secure all available data taken previously in that particular area and could then reduce its own work to a minimum.

All real estate parcels were numbered first with the key number of the sheet on which the largest part of its area occurred and then with a second number that was one of consecutive numbers on each individual sheet. On an abutting sheet, a portion of the parcel occurring on that sheet took the same number as that on the sheet on which its largest area occurred. Thus, there was no duplication of parcel numbers in the entire area and the location of any parcel of land could be quickly determined if its sheet number had been assigned.

Distances on real estate maps were scaled from the original record map except in the case of small or village areas where the field survey distance was entered. Areas for the most part were determined by planimeter, or were calculated by scaling from the record map. Each sheet had an absolute theoretical

area included within its limit, and in the determination of individual areas it was made certain that the sum total of all areas equalled the theoretical area of the sheet.

Photography

All photographic work, including the developing and printing of photographs was executed by the field office. Photographs were of standard size (5 in. by 7 in.) and for record purposes they would mounted on white cloth. Photographs were made of all buildings or other objects about which information was desired for record of real estate value. Each cemetery lot occurring within the area subject to removal was photographed to make a record of monuments and markers in existence in each individual lot.

Photographs were numbered consecutively as taken, but, for purposes of identification, a small blackboard was placed in the view. On the board the name of the property owner and the date and number of the photograph were written in white chalk. The record sheet number on which the survey of the building appeared was also shown on the board, thus tying the location down to a comparatively small area. A small white or black arrow, depending on the ground color, was placed in front of the board in the direction of the north point as determined by a pocket compass. A small sketch was drawn in the lower left-hand corner of the negative, showing the position of the camera and objects in the photograph, with relation to highways or other general features in the vicinity of the objects photographed.

Personnel

All the work of surveying and mapping, as far as the Swift River Valley is concerned, was undertaken from one field office with a personnel varying from a maximum of forty-five men to an existing field engineering force of twenty men. The work is being done by the Metropolitan District Water Supply Commission, with Headquarters Engineering Offices in Boston, Mass., all work being under the direction of Frank E. Winsor, M. Am. Soc. C. E., as Chief Engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

RAINFALL CHARACTERISTICS AND THEIR RELATION TO SOILS AND RUN-OFF

Discussion

BY C. S. JARVIS, M. AM. SOC. C. E.

C. S. JARVIS,[1] M. AM. SOC. C. E. (by letter).[1a]—The generous response in the way of discussion and constructive criticism from members of the engineering and other scientific professions has brought forth essential items or phases of the subject which no single viewpoint could encompass.

In spite of earnest endeavors to condense data to the utmost limit compatible with reasonable use, the tabulation was regarded by some reviewers as entirely too voluminous for reproduction. It is evident from occasional comments within the discussions, and likewise freely admitted by the writer, that improvement might have resulted from extending the number of stations. The 820 meteorological records and subsequent interpolations listed in Table 1 (Appendix I) were selected from 10 000 or more consulted. In an effort to be consistent, the number of representative stations chosen from among highly developed and populous countries was stringently reduced from those available, while careful gleaning of all fragments of data was required in undeveloped regions, such as the deep interior of Africa, in order that any semblance of a comprehensive view could be provided.

The great danger of "deployment in a thin line to traverse a wide front" is that the individual responsibility is thereby increased, and the danger of wide gaps intensified. Continuing the analogy, the relatively few rainfall stations selected at wide intervals to represent groups in adjacent territory, failed in some instances to reflect the entire range of behavior or precipitation patterns. However, doubling or trebling the volume of data in Table 1 (Appendix I) would not have eliminated this inherent weakness entirely, due to sparsity of numbers, and would have rendered it somewhat unwieldy without commensurate returns in the quality of information.

Mr. Bliss evidently recognizes the preliminary steps leading up to the design and formulation of projects for flood control; namely, evaluation of

NOTE.—Authors closure. The paper by C. S. Jarvis, M. Am. Soc. C. E., was published in January, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: May, 1930, by Messrs. George S. Bliss, Brehon B. Somervell, Jesse W. Shuman, and Clinton L. Bogert; August, 1930, by Messrs. C. R. Pettis, R. W. Powell, and Glen N. Cox; and September, 1930, by George C. Danforth, M. Am. Soc. C. E.

[1] Prin. Hydr. Engr., U. S. Engr. Office, War Dept., Washington, D. C.

[1a] Received by the Secretary, December 11, 1930.

rainfall intensities, duration, and intervals between probable recurrences. These, with other influences, are, after all, translated by natural phenomena into such stern realities as river gauge heights and corresponding velocities, discharge, and destructive dynamic effect.

Major Somervell points out the errors into which one might be led through neglect of the local station records. No method has yet been devised to supplant the use of all available data for a given problem. The comparatively long and complete record of Washington, D. C., so nearly identical with those of Baltimore, Md., and intervening stations, was chosen to represent the coastal plain section; while the Elkins, W. Va., meteorological data seemed to typify behavior of the mountainous region from which the head-waters of the Potomac and Monongahela Rivers flow. Neither was intended to represent an average value for the water-shed, nor an extreme of range. It is only by coincidence that the average annual rainfall determined by stream survey investigations for the Potomac River Basin should agree so closely with the mean published for the National Capital.

It is not at all surprising that extremes of annual rainfall at other stations should go beyond the range of the Elkins and Washington records. Bayard, W. Va., at Elevation 2 700, and with annual mean of 53.99 in. for 22 years of record extending intermittently to include 1929, should normally record a correspondingly higher maximum than that for Washington in nearly the ratio of their respective means, 40.5 to 53.99 in., in a given period. By proportion, 40.5 : 53.99 : : 61.3 : 81.7, showing that the maximum of 89.01 in. recorded for Bayard during 1926 is not far beyond the normally expected range for a century.

The minimum of 12.71 in. for 1895 at Western Port, Md., is probably in error, as is made quite evident by a comparison with either the few near-by stations, or the several in the surrounding country listed in Table 10. Other circumstantial evidence against the reported rainfall during 1895 at Western Port accrues from the fact that this was the first full year of record, and was subject to the disadvantage of local inexperience regarding standard methods; the recurrence of four annual rainfall depths, next in order as minima during a 35-year record, ranging between 26 and 27 in.; and the surprising deficiencies, month by month throughout most of the year, as compared with the near-by stations of Cumberland, Boettcherville, and Burlington (six-month record only), which occupy comparable positions on the terrain, although at somewhat less elevation.

Whether the Western Port minimum annual precipitation of 12.71 in. is regarded as an error in observation, or only a local freak of a sub-normal rainfall, it cannot be regarded as representative over any considerable section of the Potomac River water-shed; it would be more consistent to base judgment on a group of meteorological stations, or selected ones which represent their average behavior.

The principal value of such summaries as comprise Table 1 (Appendix I) is in furnishing a background against which the detailed data of local stations may be more broadly interpreted. No claim has been made that such a summary will supplant local information; instead, it calls for all the information

TABLE 10.—COMPARISON OF ANNUAL RAINFALL DEPTHS*

Stations	Elevation above sea, in feet	Years of Record		Mean annual rainfall	Observed Rainfall During Designated Years										
		Number	Last year included		1890	1891	1892	1893	1894	1895	1896	1897	1898	1899	1900
Morgantown, W. Va	1 250	45	1920	43.73	69.54	54.65	38.59	32.43	31.05	28.15	41.56	42.77	52.12	42.53	39.43
Rowlesburg, W. Va	1 375	36	1920	47.54	56.46	49.05	39 91	36.26	45.34	31.79	41.09	49.34	44.33	47.64	36.60
Buckhannon, W. Va	1 411	24	1920	50.12	67.55	57.29	44.75	41.88	38.07	35.19	52 85	50.51	61.14	.....	.....
Burlington, W. Va	875	29	1924	35.52	.....	.....	.....	.....	27.75	28+ †	42.22	32.95	40.78	36.58	27.67
Martinsburg, W. Va	435	29	1920	37.59	.....	.....	29.47	38.11	33.66	28.98	31.78	35.34	38.60	33.76	30.65
Harper's Ferry, W. Va	244	31	1920	36.67	36.60	31.83	33.93	30.65	30.32	29.80	34.33	43.25	44.48	39.93	34.98
Western Port, Md	1 000	35	1929	34.60	.....	.....	.....	.....	.....	12.71‡	37.18	32.61	42.90	36.84	30.64
Deer Park, Md.	2 457	24	1918	44.85	.....	.....	.....	.....	.....	29.50	49.81	44.74	53.12	37.92	36.21
Oakland, Md	2 461	20	1920	45.66	.....	.....	.....	43.97	44.86	40.66	.....	.....	.....	.....	.....
Boettcherville, Md	780	14	1905	35.31	.....	.....	34.95	37.85	32.60	28.20	43.20	35.30	38.06	28.69	28.43
Cumberland, Md	623	56	1929	35.08	52.42	46.29	34.66	30.04	28.10	23.03	37.54	35.92	41.14	34.18	28.85
Grantsville, Md	2 351	26	1920	42.48	.....	.....	.....	.....	.....	34.16	50.75	45.92	55.52	39.86	36.00
Hancock, Md	455	10	1905	35.56	.....	.....	.....	.....	.....	28.21	35.19	.....	39.37	33.16	30.30
Sharpsburg, Md. (localizing about Western Port, Md.)	420	10	1904	36.49	.....	.....	.....	.....	.....	26.76	33.18	38.57	39.94	36.56	29.32
Boettcherville, Md	780	14	1905	35.31	.....	.....	34.95	37.85	32.60	28.20	43.20	35.30	38.06	28.69	28.43
Cumberland, Md	623	56	1929	35.08	52.42	46.29	34.66	30.04	28.10	23.03	37.54	35.92	41.14	34.18	28.85
Western Port, Md	1 000	35	1929	34.60	.....	.....	.....	.....	.....	12.71‡	37.18	32.61	42.90	36.84	30.64
Burlington, W. Va	875	29	1924	35.52	.....	.....	.....	.....	27.75	28+	42.22	32.95	40.78	36.58	27.67

* With such consistent relations between the records at Western Port, Md., and neighboring stations, in all other years, it appears that the 1895 record is in error.
† At Burlington, January to June, 1895, total depth of 14.91 in.; no record for remainder of 1895; probably annual rainfall for entire year exceeded 28 in.
‡ First year of record; probably in error; rainfall for the year may have exceeded 20 in.

available, and assists in utilizing fragmentary data. A broad view of a field of research should precede the detailed study.

Mr. Shuman brings forth in attractive form the claims for periodicity of rainfall variations with relation to sunspot numbers. No one can doubt that series of years may differ in total precipitation; but the abrupt manner of changing from one extreme to another supports the impression that a portion of the normal rainfall that was temporarily withheld was later released; or a reserve somewhat depleted by unusual expenditures must await recuperation before resuming normal habits.

The comment from Mr. Shuman's discussion to the effect that, "if too large an area is taken, the data at one end or part of the district may neutralize those of another section, and one may secure no recognizable cycles," displays the weakness of the claims for sunspot influence. It reduces the coincidences to local happenings entirely, as the writer sees the problem. Ever since reliable records were initiated, there have been areas of either excessive or deficient rainfall reported; and these again have years of above normal or below normal deliveries for their respective localities. The Eastern States suffered from drought during the summer of 1930 while various other sections of the country had unusually heavy precipitation, and floods in various parts of Europe were wreaking great destruction.

If an investigator has the right to bring station records into apparent agreement of phase by assigning lags of up to ten years, then it is freely admitted that the crests and troughs of rainfall graphs may be made to coincide, for occasional short sections, with the various cycles devised, while other stations are in opposition. Why not accept the conclusions promulgated by careful observers after diligent research—that sunspot increase was accompanied or followed by increase of precipitation on about one-third of the land surface, and by a decrease on nearly an equal area; while, for the remaining third of the earth's surface, no conclusive data were available. The presumption is that either claim may be proved or disproved by nearly one-half the data, which to the writer indicates that there is "no direct relation traceable between rainfall and sunspot periodicity." If such a relation exists, as a number of reputable scientists earnestly believe and seek to establish, it is either too elusive or too thoroughly masked by local influences for definite identification at this time.

Although the pluviographs in Fig. 2 depict the behavior of only nine widely separated stations, the preliminary studies included platting of more than one hundred of the longest records chosen from all over the earth's surface, and there were nearly as many oppositions as agreements among the crests and troughs, in comparison with sunspot numbers, whether the latter are platted inverted or above the base line and whether annual rainfall for consecutive years or the progressive two-year means for the entire periods are platted.

The profession is indebted to Mr. Shuman for the clear exposition of his views on periodicity.

Mr. Bogert has pointed out how the value of the tabulation might be increased by designation of the time of occurrence for maxima and minima, both yearly and monthly. Such information could be used to advantage; but

obviously it calls for another tabulation containing such supplemental data. An attempt was made to record such dates in the original compilation; but the encumbrance entailed thereby seemed to outweigh any immediate advantage.

Professor Powell has contributed interesting and valuable views on the subject of periodicity, with reference to the recurrence of maxima or minima of a given order. The writer recognizes the alternation of sub-normal, medium, or excessive rainfall years, either singly or in groups, as they happen to occur. A season of excessive precipitation for one locality occurs simultaneously with droughts in other districts. It seems to be largely a question of routing; a large number of natural forces and elements jointly determine the path traversed by the atmospheric currents on which procurement, transportation, and delivery of moisture depend. To extend the analogy with other supply problems, the first few thousand feet of atmospheric strata above the surface of the globe, whether land or sea, high or low in either elevation or latitude, represent the loaded vehicles; the counter-currents at higher levels represent "the return of the empties", or the vehicles of reduced capacity.

The foregoing view is supported, if not established, by the physical data of Table 3, which shows that, with normal distribution or gradation of temperature with altitude, and the corresponding moisture capacity, the lower 5 000-ft. depth of atmospheric strata, if at saturation, contains considerably more moisture subject to precipitation than all the combined layers above. That relation holds, no matter whether the reckoning begins at sea level, or from mountains of any known height.

The forces which are accountable for tidal phenomena and ocean currents should extend a kindred influence on atmospheric movement. A tidal range of about 3 ft. occurs in mid-ocean; a range of 7 ft. is not unusual along extensive coast lines; more than twice this amount, occasionally quadruple, occurs under special conditions. Usual ocean tidal phenomena correspond to ordinary barometric pressure variations, measured by ½ in., or so, of mercury column at official meteorological stations, if due account is taken of the higher mobility of gases and the greater weight of water, that is, a 34-ft. column balancing the entire atmospheric pressure over an equal area at sea level.

For the extremes of observed readings, about 27.6 and 31.4 in. at the same datum, showing a total variation of more than 12%, the analogous ocean tidal range is 30 ft., or more, which may represent a considerably greater proportion of the effective offshore depth of water than the 12% range of barometric pressure noted in the foregoing remarks.

The alternation of high and low tides throughout the day seems to have its counterpart in atmospheric movements producing recurrent showers, such as those so often observed near noon or mid-afternoon, depending somewhat on latitude and the sun's declination. In desert regions, the increased convectional circulation during midday is readily traceable by the numerous whirlwind eddies, bearing aloft bits of leaves or other light substances, in a veil of dust, and displaying the main characteristics of cyclones in miniature, except that there is seldom attendant precipitation. When the desert heat has

mounted under the blazing sunshine of successive cloudless days, and has approached the limit of human endurance, the most positive and pronounced relief may follow a violent thunder shower or dust storm. The first warning is likely to be a dust cloud on the far horizon, riven by lightning flashes; then a dull roar, increasing momentarily and punctuated by occasional thunder. In contrast, the prevailing calm and stillness are almost ominous until the moving curtain is within perhaps less than a mile of the observer, when he must suddenly combat fitful wind and cross-currents, followed by a murky cloud raised from the sweeping of the desert floor; then, infrequently, a brief deluge of rain, fiercely driven or occasionally interspersed with hail; and, finally, a few moments after the drenching, a cool, cleansed, and freshened atmosphere rapidly resumes the calm such as preceded the storm.

If the analogy continues to hold between atmospheric and ocean tides, there should be a fortnightly recurrence of unusually energetic circulation of the atmosphere, corresponding to spring tides at seaports. Whether or not precipitation accompanies such disturbances, depends largely on relative humidity and the temperatures that prevail locally; in short, on the climatological habits. A systematic examination of prominent long-record meteorological data, day by day throughout any month or series of months, ordinarily discloses pronounced periods of maxima in wind movement, and, less frequently, in rainfall, occurring at fortnightly intervals or less. Secondary disturbances interspersed probably account for some of the occurrences.

From a consideration of evaporation rates from either free water surface, wet ground, or dense foliage, together with atmospheric moisture capacity, relief by condensation at intervals of a month or less may be regarded as unavoidable at some place along the path traversed, unless intermixed with air currents deficient in humidity. Whether the evaporation rate is the usual 4 in., or more, for months of moderate temperature, or less than 1 in. for a winter month in temperate zones, the corresponding moisture capacity of the entire atmospheric column would not be greatly in excess thereof for average prevailing temperatures. Under such conditions it appears that the initial steps in precipitation would begin at the nearest barrier, whether coast line, mountain range, or mass of dense, cold air obstructing the path of the warmer, moisture-laden currents.

After several hours of intense sunshine, without the moderating effect of free water for evaporation, the upward displacement of warm air to re-establish equilibrium doubtless produces an effect comparable with high tide in sea movements, and thereby affects rainfall distribution nearly as definitely as do the physical obstructions in mountainous regions.

A large number of cross-sections across mountain ranges discloses a fairly definite relation with respect to slopes. It appears that the abrupt changes in gradient also mark pronounced variations in rainfall. The top of the steepest slope, the highest point from which the entire lower slopes can be viewed (known in tactical problems as the military crest), seems to receive the greatest precipitation; and the actual summit may record a considerable reduction in amount. The leeward slopes with respect to moisture supply

almost invariably receive deficient rainfall because of the increase of moisture capacity with temperature during the descent.

Professor Cox has pointed out such defects in the platting of isohyetals in Fig. 5 as must inevitably occur, in local sections, if only a few selected stations are utilized. On this account, General Note No. 3 of Fig. 7 is equally applicable to related Figs. 5 and 6, and was so intended, reading as follows: "This map is subject to revision by addition of data covering local variations not considered in Table 1".

It should be observed that the data from Table 1 (Appendix I), for Iowa stations, with one exception, terminated with 1920. Up to that time, the infrequent, sporadic occurrences of more than 7 in. in 24 hours, were so out of line with the usual habits that it appeared inadvisable to let those isolated data govern the platting in opposition to the general habits. Without exception such rainfall was the result of thunder showers during summer or adjacent months and confined to restricted areas entirely away from the longest established and best equipped stations. Such problems demonstrate the importance of introducing the dimension or element of periodicity for interpreting and classifying observations of natural phenomena, as illustrated in Table 11 for annual rainfall.

From the new data contributed by the discussion, it appears that either an unusual number of intense 24-hour rainfalls have occurred in Iowa during the past decade, or else the list of stations must have been extended recently.

Mr. Danforth expresses an appreciation of "World Weather Records" in which the writer concurs, as far as the arrangement, convenience, and value of the data are concerned. This volume was published during the writer's quest for basic information, and includes 355 precipitation records, of which 251 were incorporated in Table 1 (Appendix I), in many instances supplemented by fragmentary data from other sources, or by extensions of summations so that the new means and extremes cover the entire period of available record.

The latitude and longitude co-ordinates were retained for purposes of identification of obscure or remote stations during the process of platting; but after the routing and the plotting of all stations on Fig. 7, the use of an entire line across two pages for such co-ordinates seemed to be out of proportion to the service that could be rendered thereby. Whenever the question of exact location enters into the problem, the length of record, the altitude of a station, and the rainfall patterns into which both the annual and the monthly means and extremes readily classify, should identify the particular station listed, even if there are others of the same designation, as in the case of two named Alto de Serra, in Brazil.

The use of such familiar names as Pernambuco and Para to indicate the principal ports of States bearing the same names, although technically incorrect, seems to be almost universal in news items, and extends to such standard volumes as "The Climate of the Continents",[3] by W. G. Kendrew.

[3] Oxford, Clarendon Press, 1922.

Colonel Pettis has given evidence of broad experience and understanding concerning flood-flow phenomena and their relation to rainfall. The supplemental data for 24-hour periods at ten of the Colorado stations are gratefully acknowledged.

The question of determinate limits as used in the original paper was not intended to go beyond certain considerations, either axiomatic, logical, or demonstrable in the light of established facts. However, the discussion of typical curves derived from a study of probabilities is both interesting and instructive.

Conclusions.—For purposes of classification, it appears logical to compare rainfall distribution, either seasonal or annual, with normal evaporation from free water surfaces for the same periods. Where the potential losses from evaporation are approximately equal to the normal precipitation, the needs of ordinary crops on loamy or other fine-textured soils are abundantly supplied without recourse to artificial methods of application during average years.

Where rainfall is habitually greatly in excess, the over-supply is manifested in the marshy character of both the soil and the vegetation, unless either surface or sub-soil drainage is unusually well developed. The regulating effect on run-off thereby produced is generally accentuated by the attendant habits of frequent gentle precipitation.

In contrast with the superabundant rainfall is the deficient rainfall, amounting to decidedly less than normal evaporation depths from free water surfaces, and ranging anywhere below, say, two-thirds of that amount. In a general way, the rainfall habits are recorded by both the surviving vegetation and the soil structures developed under varying degrees of aridity. The limits of plant rootage and moisture percolation, saturation, oxidation, leaching, and deposition are well marked by horizontal stratification and gradation of coloring, or by the widespread calcareous accretion in subsoil known as caliche or hardpan. The latter conserves to the utmost the sparse moisture supply, and hastens saturation or water-logging when it is supplemented by artificial means or by seasons of abnormally high rainfall.

With due regard to the foregoing definitions, the 15-in. average annual precipitation prevalent over extensive areas of Alaska, Canada, Greenland, Northern Europe, Siberia, and parts of the high Andes Mountain range is comparable with approximately a 40-in. depth over the Middle Atlantic coastal plain, or with nearly double this rainfall in tropical lowlands; and, in each instance, the annual precipitation and evaporation should nearly balance. As progressive increase in altitude along a high mountain slope nearly reproduces the climatological and soil conditions of higher latitudes, the elevations above sea level for meteorological stations supply some of the primary factors in the study of rainfall and run-off; also, in the classification as to aridity or humidity of climate as determined by requirements for plant growth. For simplicity, the three principal classes previously defined may be regarded as those with deficient, moderate, or optimum (in technical terms for "ideal" or "most favorable" conditions); and, finally, excessive precipitation.

TABLE II.—EXTREME AND MEANS OF ANNUAL PRECIPITATION FOR VARIOUS PERIODS

(Based on Analysis of Existing Records Selected at Random for Length and Geographical Distribution)

Serial number from Table I (Appendix I)*	Station	Years of record	Expressed, in Inches						Expressed, in Percentage					
			Minima			Mean	Maxima			Minima		Mean	Maxima	
			100 years	50 years	10 years		10 years	50 years	100 years	100 years	50 years	10 years	10 years	50 years	100 years
15	Burlington, Vt.	93	21	24	28	82.6	36	43	49	64	74	86	100	111	132	150
23	Boston, Mass	110	27	30	34	48.5	52	60	67	62	69	78	100	120	138	154
39–A	New York, N. Y.	101	29	31	34	42.1	48	55	59	69	74	81	100	114	131	141
43	Albany, N. Y.	101	26	28	30	37.4	42	52	57	69	75	80	100	113	139	153
58	Philadelphia, Pa.	110	29	31	35	42.4	49	56	61	68	73	83	100	116	130	144
69	Pittsburgh, Pa.	91	25	26	30	36.2	41	44	51	69	77	83	100	113	121	141
76	Washington, D. C.	89	19	24	32	40.6	49	58	61	46	59	79	100	121	143	150
97	Charleston, S. C.	136	24	31	33	47.5	62	70	78	51	65	70	100	131	148	165
109	Key West, Fla.	78	21	24	30	37.9	49	56	70	55	63	79	100	129	148	185
119	New Orleans, La.	81	31	39	44	57.0	68	81	86	54	69	77	100	119	142	151
139	Little Rock, Ark.	48	.....	31	35	48.2	62	76	.....	.....	65	73	100	128	158	.....
148	St. Louis, Mo.	91	23	26	31	39.7	49	61	69	58	65	78	100	124	154	174
167	Chicago, Ill.	93	22	24	27	33.1	40	44	47	66	73	82	100	121	134	142
171	Cincinnati, Ohio.	93	18	24	30	40.7	49	55	65	44	66	74	100	121	135	160
185	St. Paul, Minn.	90	10	16	21	27.2	35	40	50	37	57	77	100	129	148	184
204	Seattle, Wash.	52	.....	22	27	33.6	43	56	.....	.....	65	80	100	128	167	.....
226	San Francisco, Calif.	80	9	10	15	22.3	30	37	39	40	45	67	100	135	166	175
236	San Diego, Calif.	80	3	4	6	9.6	14	19	28	31	42	63	100	146	198	292
258	Sacramento, Calif.	80	8	9	12	18.3	23	30	35	44	49	66	100	126	164	191
273	Salt Lake City, Utah.	55	.....	10	12	16.3	21	24	.....	.....	61	74	100	129	148	.....
336	Colon, Panama	57	.....	86	103	127.3	154	188	.....	.....	68	81	100	121	144	.....
377	Rio de Janeiro, Brazil.	75	.....	29	32	43.2	55	65	.....	.....	67	74	100	127	151	.....
401	Havana, Cuba.	67	.....	28	41	48.2	69	65	.....	.....	58	85	100	123	135	.....
416	Sitka, Alaska.	64	.....	58	70	84.8	102	140	.....	.....	69	88	100	121	166	.....
468	Sydney, Australia.	85	21	24	35	47.9	63	81	83	44	50	73	100	132	170	174
498	Tokyo, Japan	45	.....	46	52	59.9	77	86	.....	.....	77	87	100	129	144	.....
516	Calcutta, India	92	39	43	52	64.3	76	89	93	61	67	81	100	118	138	145
521	Madras, India.	108	18	22	34	49.6	70	80	88	36	44	69	100	141	161	177
567	Capetown, South Africa.	87	17	19	21	25.3	31	36	41	67	75	83	100	122	142	162
618	Beirut, Syria.	45	.....	23	27	35.5	46	51	.....	.....	65	76	100	130	144	.....
649	Milan, Italy.	161	25	27	30	39.4	49	54	62	64	69	76	100	125	138	158
651–A	Padua, Italy.	202	19	21	25	33.8	44	56	62	56	62	74	100	130	148	183
654	Rome, Italy.	143	13	20	26	32.7	41	45	58	40	61	86	100	126	138	177
662	Lyons, France.	80	19	20	24	29.1	35	39	45	65	69	83	100	120	134	155
665–A	Paris, France.	214	9	11	14	19.9	25	28	30	45	55	70	100	126	141	151
695	Greenwich, England.	81	16	18	20	24.3	30	34	36	66	74	82	100	123	140	148
748	Warsaw, Poland.	109	15	16	18	21.2	27	31	46	71	75	85	100	128	146	217
753	Leningrad, Russia.	95	12	15	17	19.2	24	27	29	63	78	89	100	125	141	151
797	Helsingfors, Finland.	80	14	15	19	24.4	30	32	34	57	61	78	100	123	132	139
807	Copenhagen, Denmark.	100	14	16	18	22.5	27	29	30	62	71	80	100	120	129	133

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, pp. 32 *et seq.*

The temperate days and cool nights with heavy dew, so prevalent in the uplands as well as in high latitudes, reduce the demands for moisture measured as rainfall. This fact plus the advantage of strategic position, with priority privileges, so to speak, accorded to high mountain slopes, re-acts to make the uplands relatively immune to drought, while the lower interior valleys and plains are the first to suffer depletion, and the last to be served anew. High mountain peaks are swathed in fog, mist, or light rain, usually for hours both before and after the downpour that reaches the lowlands. The greater heights through which the droplets fall to reach the valleys afford opportunity for increased diameter of raindrops, velocity of descent, amount of electric potential induced thereby, further coalescence through attraction of opposite charges; and, finally, the lateral displacement of condensed moisture from its normal path of descent by cross-winds, producing an effect similar to the folding of curtain draperies, and resulting in multiplied intensity of rainfall in some parts of the storm area at the expense of depletion for others, located to windward.

Such considerations account in part for the gentle, persistent rainfall of humid areas as opposed to the irregular, fitful, and violent phenomena of arid or semi-arid sections. In terms of stream flow or water supply, there is the discrimination between primary and secondary rights, with their contrasts of dependability. Whether the favored strategic position depends for its advantage on altitude, or on proximity to either the source of atmospheric moisture or its main lanes of traffic, or on the abrupt steepening of gradient to surmount a barrier, the beginnings of condensation merge almost imperceptibly with fog for part of the time.

If the wind movement becomes unusually strong, or if the energy released by the process of condensation is transformed into the kinetic phase to act as a propellent, then the storm may carry over the leeward side of the barrier, whether it is a land form, an underflowing mass of dense atmosphere, or merely the impediment of barren distance, for the relatively brief period of greatest activity and intensity.

The high ratios between successive values of either rainfall or percentage as given in Table 1 (Appendix I), are apparently a fair measure of relative aridity, whether the daily, monthly, or yearly periods are the basis of reckoning. For either moderate or excessive precipitation stations, either the depths of rainfall or the percentages of the mean annual which they represent (as tabulated), make fairly regular ascending series when read from left to right. For stations with deficient rainfall in comparison with potential evaporation from free water surfaces, the series are broken by occasional inversions or reversals. For example, the maximum monthly rainfall may far exceed the minimum annual, instead of being approximately one-half or two-thirds of the value in the next column, as seems to be characteristic of stations with dependable and ample rainfall. Furthermore, for stations of moderate rainfall, a typical 50-year record may show minimum and maximum percentages of about 50 and 150, respectively, or, more often, 60 and 140. For excessive rainfall stations, 70 and 130% may be more nearly representative, while for regions of marked

deficiency of rainfall compared with potential evaporation, the range may be 40 and 200%, or, more rarely, nearly 30 and 300 per cent. Moreover, the average 10-year minima and maxima should ordinarily fall almost intermediate between the 100% and the foregoing extremes, although there may be lack of symmetry for some records, and decided departure from arithmetical series for others. The first approximations for 10-year minima and maxima for all the foregoing cases, and, in the same order, would therefore be 75 and 125; 80 and 120; 85 and 115; 70 and 150; and, finally, 65 and 200%, respectively. Table 11 displays results of further studies along this line. Some of the irregularities may have resulted from varying reliability of either the rainfall data or the results of periodicity analysis.

As may reasonably be expected, the periodicity relations dealing with monthly rainfall display the same general trends and characteristics, but involve higher ratios. A fairly promising field awaits development along the same lines, but will require organized effort and facilities.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

COMPLETION OF MOFFAT TUNNEL OF COLORADO

Discussion

BY CLIFFORD ALLEN BETTS, M. AM. SOC. C. E.

CLIFFORD ALLEN BETTS,[1] M. AM. SOC. C. E. (by letter).[1a]—The valuable discussions have brought out some important points. Mr. Davies' reference to the limitations imposed on the Engineering Staff by legislative action and, later, by legal contracts, illustrates the interdependence of economic feasibility, sound engineering construction, and adequate provisions for repayment of cost, all of which are essential to the ultimate success of projects in general and of this one in particular.

The merits of electrification, especially where a heavy volume of traffic is involved, are indicated in Mr. Keay's outline of European practice, which he has had an unusual opportunity to study. Electrification of the Moffat Tunnel had been taken for granted up to the time the lessee exercised the option of steam operation. The capacity of the tunnel has been estimated at 40 trains daily. This use is contingent upon utilization of the Dotsero Cut-off for which the way was cleared in 1930 by the United States Court decisions validating the lease with the Denver and Salt Lake Railroad Company and by the Interstate Commerce Commission's approval of the acquisition of the Denver and Salt Lake Railway by the Denver and Rio Grande Western Railroad Company.

The effect of steam operation on this partly timbered, 6-mile, tunnel will be watched with interest in order to determine whether the chemical action of hot flue gases and the subsequent forced ventilation drafts accelerate deterioration. In this connection, the statement by Mr. Eremin that "a serious disadvantage of timber lining is the difficulty of preserving it from decay", is apropos and explains the recommendation that a concrete-lining program be carried out as maintenance replacements of timber become necessary. This would permit deriving the maximum use of the investment in

NOTE.—Author's closure. The paper by Clifford Allen Betts, M. Am. Soc. C. E., was published in April, 1930, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: September, 1930, by Messrs. J. Vipond Davies and R. H. Keays; and November, 1930, by A. A. Eremin, Assoc. M. Am. Soc. C. E.

[1] Engr., U. S. Bureau of Reclamation, Owyhee Dam, Nyssa, Ore.

[1a] Received by the Secretary, December 24, 1930.

treated timber lining, would defer large interest charges on concrete, and would provide an opportunity to study the unusually severe ground pressures prior to concreting the remainder of the bore.

There is a wide range in physical and chemical properties of ground. The Moffat Tunnel contained numerous formations from blocky, solid rock unaffected by weathering to a squeezing and swelling fault zone material that produced upward pressures in the tunnel floor as well as top and side pressures. This type of ground (which might be added to the two illustrated by Mr. Eremin in Fig. 13) is difficult to hold because the flowing pressures find the weakness of the lining whether it be by shear at the joints or by bending of segmental arch ribs or posts. In shortening timber arch members to approach a circle the stability of the arch is endangered, but if steel members are used, the circular arch provides most effective resistance to such pressures. This is being borne out in the water-tunnel lining.

An excellent opportunity to substantiate Mr. Eremin's statement that "pressure on the tunnel lining varies with the size of the tunnel and the arching effect of the ground", occurs in two tunnels with which the writer is connected. A 3.55-mile water tunnel, of the horseshoe type, 16 ft. 7 in. in diameter, is being driven at approximately right angles to a 4.15-mile tunnel, 9 ft. 3 in. in diameter, with only 300 ft. separating the inlet portal of one from the outlet of the other. Where similar lava and tuff are encountered in each, the smaller tunnel stands without support, while the larger bore requires 10 by 10-in. fir timbers on 5-ft. centers to hold the ground until the concrete is placed. The difference in pressure between the two tunnels is very noticeable. Advantage can be taken of this fact in determining the economic balance between cost of lining and the excavation costs in designing tunnel sizes.

Recent developments on the Colorado River emphasize the strategic location of the Moffat Water Tunnel for diversion of a water supply for Denver, Colo., from the head-waters of that stream. Due to the rapidly increasing value of water in the arid West and the natural limitations of the watersheds available for serving Denver by gravity, this water tunnel has great potential value.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FREQUENCY AND INTENSITY OF EXCESSIVE RAINFALLS AT BOSTON, MASSACHUSETTS

Discussion

BY CHARLES W. SHERMAN, M. AM. SOC. C. E.

CHARLES W. SHERMAN,[1] M. AM. SOC. C. E. (by letter).[1a]—The ideal which every designer of storm-water conduits undoubtedly has in his mind, has been admirably expressed by Mr. Horner. However, it is not only improbable that any designing engineer has made such a statement, but unlikely that it ever can be made. Nevertheless, sufficient information has now been accumulated to make possible a somewhat closer approximation to the ideal than has usually been attempted.

The writer also had a vision, but it extended only to the possibility that one type of rainfall intensity equation might be found to apply to a large section of the country—perhaps to the portion east of the Rocky Mountains—and that the frequency relation might be the same for all localities. If this were so it would be necessary only to determine the value of K_1 (in which, K corresponds to a frequency of one year), and all the necessary figures relating to rainfall intensity would be available. Moreover, analyses of precipitation records at a considerable number of stations would then make it possible to prepare a map showing lines for equal values of K_1, and interpolations between these lines would be sufficient, in most cases, to find a proper rainfall-intensity coefficient for almost any locality.

This vision may have been a mirage, and although the writer is disappointed, he is not surprised to learn that Messrs. Wagnitz and Wilcoxen do not find the same form of equation applicable at Detroit as that derived for Boston. That the frequency relation should be approximately the same in both cases, appears worthy of special notice. To be sure, the analysis of Detroit figures did not include any durations beyond 360 min., and it is possible that somewhat different results would have been obtained if longer storms had been taken into account.

NOTE.—Author's closure. The paper by Charles W. Sherman, M. Am. Soc. C. E., was published in April, 1930, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: September, 1930, by W. W. Horner, M. Am. Soc. C. E.; and December, 1930, by Messrs. C. S. Jarvis, and Milton F. Wagnitz and L. C. Wilcoxen.

[1] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[1a] Received by the Secretary, January 19, 1931.

The use of the method of least squares in computing the constants for the formulas, by Messrs. Wagnitz and Wilcoxen, appears to be advantageous, particularly in determining the best possible form of curve to fit a particular set of records. However, it appears from an examination of Fig. 3 that the form of curve resulting from these computations does not conform very well with the observed values of the first, second, and third magnitudes, and even the divergence from the points of thirtieth magnitude (1-year storm) is considerable. This may mean that for Detroit conditions the form of equation for each frequency is different from that for every other frequency. Perhaps, however, if the data were available for storms of longer duration, the apparent discrepancy between curves and plotted points would be less marked. It may be noted that if Fig. 1 had extended only to 3-hour storms, the divergence between the curves and the observations would have been more noticeable than is actually the case.

In view of the many items in storm-water design, which have to be fixed by the exercise of judgment, it is not impossible that a formula for intensity-duration relation of the form suggested in the paper may be close enough for general use; that the denominator should be $(t + 7)$, or $(t + 8)$, with an exponent of $\frac{3}{4}$, $\frac{4}{5}$, or $\frac{5}{6}$; and that the frequency relation may be taken as a function of $F^{\frac{1}{4}}$. Such figures, like the $\frac{3}{2}$-exponent in the usual weir formula, would be easy to remember and to apply, and might prove to be correct enough for ordinary use.

Mr. Horner is right in noting that the curves of Fig. 1 appear to be higher than the observations justify, for durations exceeding 30 hours, and this is particularly obvious in examining Fig. 2. A modification of the form of the curve, requiring a more complex form of equation, appears to be necessary if the points are to be approximated more closely; if, indeed, any equation can be found which will fit the observations. It is possible that the parallel curves obtained from the Boston observations represent a fortuitous circumstance, and that records at other places will not show similar results. Until the records of a considerable number of stations have been analyzed, it is possible only to speculate on this point.

Mr. Jarvis' comparison of the relations between intensities and frequencies is interesting, and is in such a form as to be easily remembered and applied. Since the Detroit data furnish some degree of confirmation of the figures obtained for Boston, these relations may be considered more likely to be of general application than any of the other figures presented. Whether the "semblance of consistency" in the intensity-duration relation is really reassuring, or just interesting and suggestive, can only be known when other records have been studied in a similar manner.

The writer agrees with Merrill M. Bernard, M. Am. Soc. C. E.,[2] that the intensity-duration formulas of the types, $i = \frac{K}{t + b}$ and $i = \frac{K}{t^a}$, are not

[2] "Formulas for Rainfall Intensities of Long Duration," *Proceedings*, Am. Soc. C. E., October, 1930, Papers and Discussions, p. 1835.

equally applicable to both long and short periods of duration; but he believes that the type, $i = \frac{K}{(t + b)^a}$, is equally applicable to both long and short durations. Obviously, when t is large and b is less than 10, the difference between t^a and $(t + b)^a$ is insignificant.

There is considerable probability of error in using the Morgan and Meyer data in the way Mr. Bernard has done in the paper cited, first, because few, if any, "1-day", "2-day", or "3-day" storms lasted exactly 24, 48, or 72 hours; and, second, because the estimation of frequencies by computations based upon the sum of the length of records at several stations is open to severe criticism. Obviously, the sum of 100 one-year records, all for the same year, at 100 stations, cannot be equivalent to a record of a century at a single station. However, taking the figures given in Mr. Bernard's Table 3[3] for Quadrangle 15-*E*, and leaving out of consideration the slight differences in exponent of the denominator, it would appear that the frequency relation would be a function of $F^{0.22}$. Possibly this may be taken as further confirmation of the idea that the frequency relation may generally approximate a function of $F^{\frac{1}{4}}$.

[3] *Proceedings*, Am. Soc. C. E., October, 1930, Papers and Discussions, p. 1842.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ANALYSIS OF CONTINUOUS FRAMES BY DISTRIBUTING FIXED-END MOMENTS

Discussion

BY MESSRS. R. R. MARTEL, AND CLYDE T. MORRIS

R. R. MARTEL,[6] M. AM. SOC. C. E. (by letter).[6a]—In his concise presentation, Professor Cross has restricted himself to a single, comprehensive, illustrative example. From this, the careful reader can grasp the two principles involved and apply them to other problems of the same general character without resorting to formulas of any kind. This method is a marked advantage over those others which require substitutions in formulas and subsequent algebraic manipulations.

The author's terse statement of the modifications by which the effect of side-sway, resulting from either unbalanced loading, or due to horizontal forces, may be evaluated, has led some of the discussers to illustrate the application of this by examples.

One such example for unbalanced loading can also be used (with very little change) to illustrate the method of dealing with horizontal forces. If the bent shown in Fig. 10, is subjected to horizontal forces of 1 000 at each floor, the procedure would be as follows:

(1) Apply any convenient arbitrary moment to the first-story columns of the bent, balance and distribute. Compute the shears in each story and, from these, the horizontal force which will produce the moments obtained after balancing and distributing.

(2) Apply any convenient arbitrary moments to the second-story columns, balance and distribute. Compute the shears in each story and, from these, the horizontal force which will produce the moments obtained after balancing and distributing.

(3) Write simultaneous equations to find the ratios of the forces obtained in Steps (1) and (2) to the given forces.

(4) Multiply the distributed moments obtained in Steps (1) and (2) by the respective ratios found in Step (3). The algebraic sum of

NOTE.—The paper by Hardy Cross, M. Am. Soc. C. E., was published in May, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1930, by Messrs. C. P. Vetter, L. E. Grinter, S. S. Gorman, A. A. Eremin and E. F. Bruhn; October, 1930, by Messrs. A. H. Finlay, R. F. Lyman, Jr., R. A. Caughey, Orrin H. Pilkey, and I. Oesterblom; November, 1930, by Messrs. Edward J. Bednarski, S. N. Mitra, Robert A. Black, and H. E. Wessman; and January, 1931, by Messrs. Jens Egede Nielsen, F. E. Richart, and William A. Oliver.

[6] Prof. of Structural Eng., California Inst. of Technology, Pasadena, Calif.

[6a] Received by the Secretary, December 9, 1930.

these two sets of moments gives the moments desired. Steps (1) and (2) are demonstrated in Figs 11(*b*) and 11(*c*).

The simultaneous equations of Step (3) would be:

$$+ 142.3\ C_1 + 295.8\ C_2 = + 1\,000 \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

$$+ 560.3\ C_1 + 369.7\ C_2 = - 1\,000 \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

from which the values of C_1 and C_2 are found to be -5.87 and $+6.20$, respectively. The operations called for in Step (4) are shown in Fig. (27).

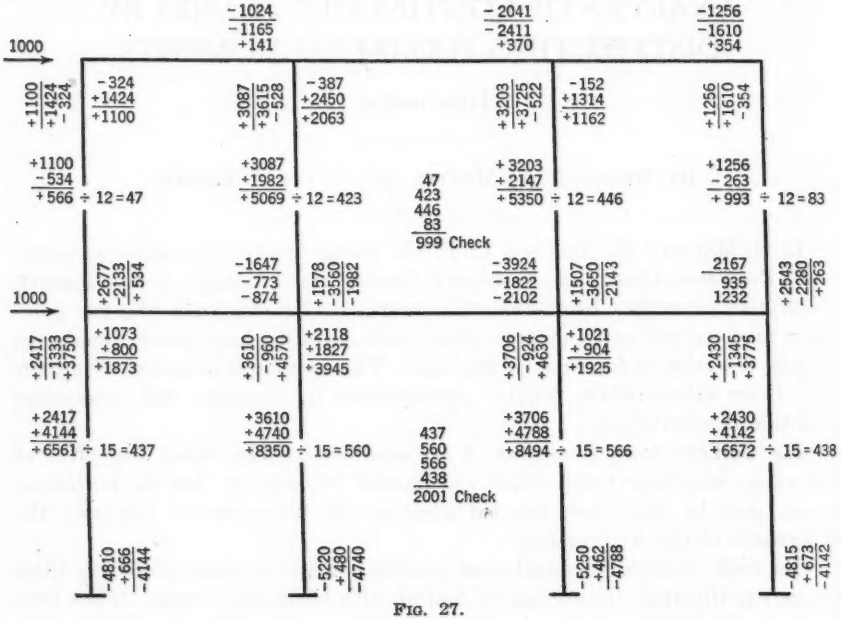

FIG. 27.

A crane column subjected to moment due to the eccentricity of the crane girder, furnishes another simple illustration of the application of the general method with side-sway.

Given such a column subjected to a moment of 243 ft-kips at the crane rail, and with end conditions which justify the assumption of perfect fixity, the lower section of the column is 36 ft. long and the upper portion, 14 ft. long; the ratio of moments of inertia of these sections is 5 : 1.

It is required to find the moments at the ends and at the crane rail:

(*a*) Assume an imaginary support, *F*, (Fig. 28(*a*)) at the crane rail and distribute the moment between the two sections of the column. Then "carry over" to the ends of the column.

(*b*) Compute the shears in both portions of the column. The difference between these shears is the value of the force, *F*, exerted by the imaginary support. Removing this support will produce moments in the columns the same as by applying a transverse load at that point to the crane column considered as a fixed-ended beam. These moments are most easily determined by a slight variant of the general method of distributing unbalanced moments.

(*c*) Assume arbitrarily a moment, say, 10, in one part of the column, which is temporarily considered direction-fixed (locked) at the crane rail, (Fig. 28(*b*)). The moment in the other section (13.2), is then obtained from the relation,

$$M_1 \frac{l_1^2}{I} = M_2 \frac{l_2^2}{I_2}$$

(*d*) Remove the restraint at the crane rail (unlock), distribute the unbalanced moments, and "carry over" to the column ends.

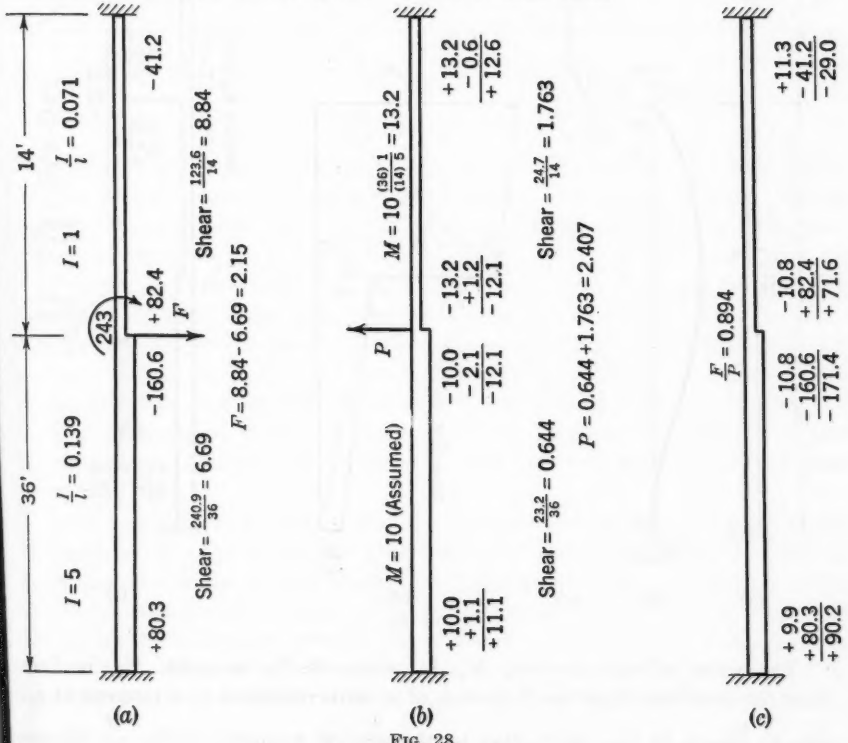

FIG. 28.

(*e*) Compute the shears in both sections of the column, and, from these, the force, P, which produces the moments obtained in Step (4).

(*f*) Multiply these moments by $\frac{F}{P}$, the ratio of the force obtained in Step (2), to the force obtained in Step (4).

(*g*) Add these moments to those found in Step (1), giving the final results. (See Fig. 28(*c*).)

Instead of being rigidly fixed at the top, the crane column may be held at two points by the top and bottom chords of a roof truss. This case can also be solved by the use of an imaginary support at the crane rail. However, it is perhaps as simple and more interesting to consider this problem as a con-

tinuation of the one previously given, since in this way there will be a span in which the moment of inertia is not constant and its influence on distributing and carrying over will be brought out.

Let the lower 50 ft. of the column be the same as before. There is an additional 7 ft. at the top between the two points of support and its moment of inertia is unity. (See Fig. 29(*a*).)

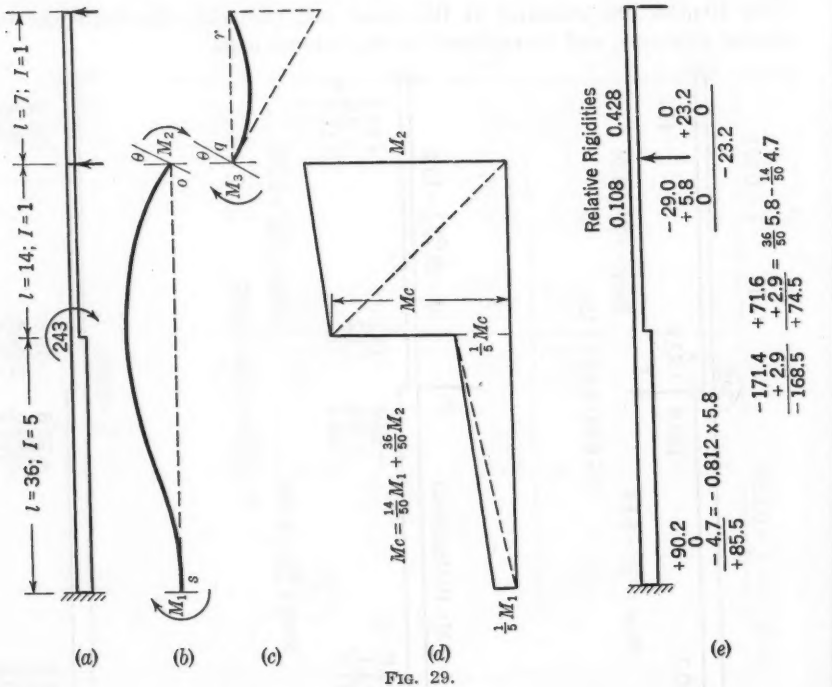

FIG. 29.

The value of the moment, M_1, in terms of the moment, M_2, is found from the condition that the deflection of o, with reference to a tangent at s, is zero, as shown in Fig. 29(*b*); that is, the statical moment of the $\frac{M}{I}$-diagram (Fig. 29(*d*)) about b is zero. The resulting equation is:

$$\frac{14}{2} \times \frac{14}{3} M_2 + \frac{38}{5} \times \frac{36}{2} M_1 + (0.72 M_2 + 0.28 M_1) \left(\frac{28}{3} \times \frac{14}{2} + \frac{26}{5} \times \frac{36}{2}\right) = 0 \quad \ldots\ldots (33)$$

from which,

$$M_1 = -0.812 M_2 \quad \ldots\ldots (34)$$

Hence, the carry-over factor for this case is -0.812 instead of $-\frac{1}{2}$.

The angle change at o produced by a moment, M_2, can now be determined from the area of the $\frac{M}{I}$-diagram. Thus,

$$\theta E = \frac{14}{2} M_2 + \frac{36}{(2 \times 5)} M_1 + (0.72 M_2 + 0.28 M_1)\left(\frac{14}{2} + \frac{36}{2 \times 5}\right) \quad (35)$$

and, $\theta E = 9.28 M_2$, or,

$$M_2 = 0.108\, \theta E \quad (36)$$

Considering the upper section, Fig. 29(c), subjected to a moment, M_3, the angle change at q, is the deflection at r, with reference to a tangent at q, divided by the length, $c\,d$; that is, $\theta E = \frac{1}{3} \times \frac{7}{1} M_3$, or,

$$M_3 = 0.428\, \theta E \quad (37)$$

Hence, for a given angle change, θ, the ratio of M_2 to M_3 is 0.108 to 0.428. This is the ratio used in distributing the unbalanced moment between the two sections of the column. The remaining operations required to obtain the final moments are shown on Fig. 29(e).

Because so little additional work is involved, it is of interest to consider the effect of roof truss deflection on the moments in the crane column. Let the column be the same as before and assume that it is subjected to an outward deflection, δ_1 (by the bottom chord of the truss), of 0.125 in. and an inward deflection, δ_2 (by the top chord), of 0.100 in. (see Fig. 30(a)).

The moments of inertia of the column will be taken at 2 000 and 10 000 for the portions above and below the crane rail, thus maintaining the 1 : 5 ratio previously used.

It is convenient to consider that the column remains direction-fixed at the lower chord, Fig. 30(b), and on this basis to compute the moments resulting from the deflections in the upper and lower portions of the column. The moments thus found on each side of the bottom chord will not be equal. Removing the imaginary restraint, by means of which direction-fixity has been maintained, will produce changes in the moments that may be evaluated by distributing the unbalanced moment and carrying over in the usual way.

For the lower portion of the column the statical moment of the $\frac{M}{I}$-diagram about n equals $E I \delta_1$. The left-hand side of the resulting equation is the same as in Equation (33) and $E I_1 = 30\,000\,000 \times 2\,000$ lb-in.^{2} $= 417\,000$ kip-ft.^{2}; $\delta_1 = 0.125$ in. $= 0.0104$ ft.; M_1 and M_2 are in feet-kips; or,

$$147.2 M_2 + 181.5 M_1 = 4\,340 \quad (38)$$

Since this portion of the column is direction-fixed at both ends, θE, in Equation (35), is zero, or,

$$14.64 M_2 + 6.57 M_1 = 0 \quad (39)$$

Solving Equations (38) and (39) simultaneously, $M_2 = -16.9$, and $M_1 = +37.7$.

For the upper portion of the column, the moment, M_3, is obtained from the relation,

$$M_3 l^2 = 3 E I_1 (\delta_1 + \delta_2) \quad \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

in which, $\delta_1 + \delta_2 = 0.125 + 0.100 = 0.225$ in. $= 0.0187$ ft., or, $M_3 = 477.3$.

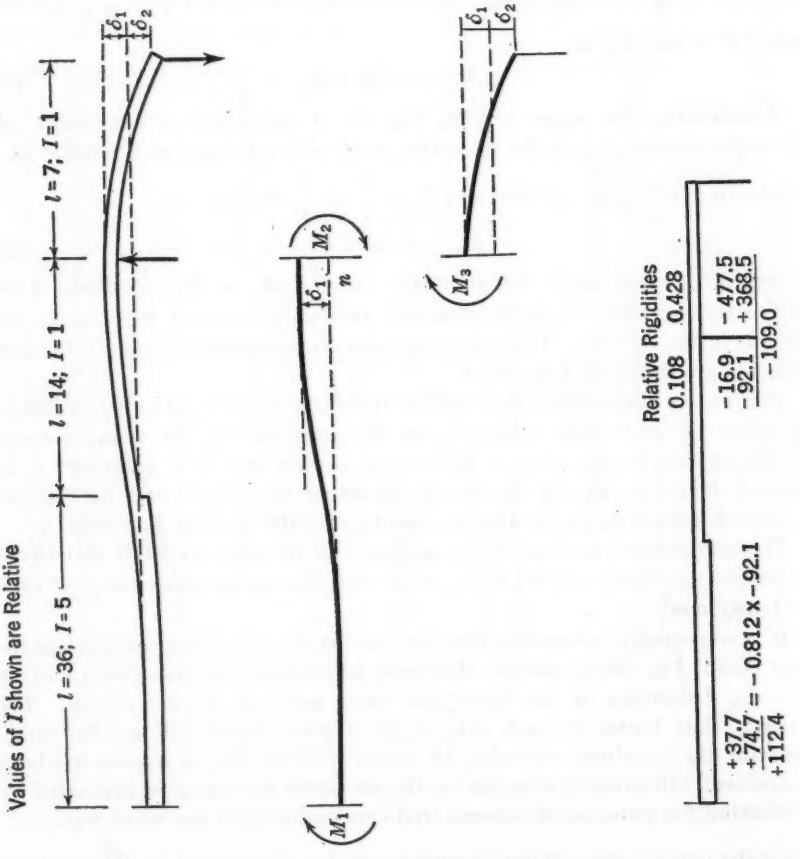

FIG. 30.

The remaining operation of distributing and carrying over are indicated in Fig. 30(*c*). It will be noted that the relative rigidities and the carry-over factor are the same as in the previous problem.

CLYDE T. MORRIS,[7] M. AM. SOC. C. E. (by letter).[7a]—The writer has found the Cross method of analyzing continuous frames of inestimable value; but the sign convention used, when applied to a multi-storied multi-paneled bent, is more cumbersome than the following: "Any moment which tends to rotate a

[7] Professor of Structural Eng., Ohio State Univ., Columbus, Ohio.

[7a] Received by the Secretary, December 30, 1930.

joint clockwise is to be considered positive, and the reverse negative".[8] After the solution is completed, it is a simple matter to translate these signs into the usual convention.

The application of the method to the solution of stresses in building frames due to wind load is not explained. Professor Cross indicates that it is possible, and several of the discussers have mentioned the effects of side-sway due to eccentric loads. The method may be extended to lateral forces on a tall building frame with as much ease and accuracy as for vertical loads in the following steps:

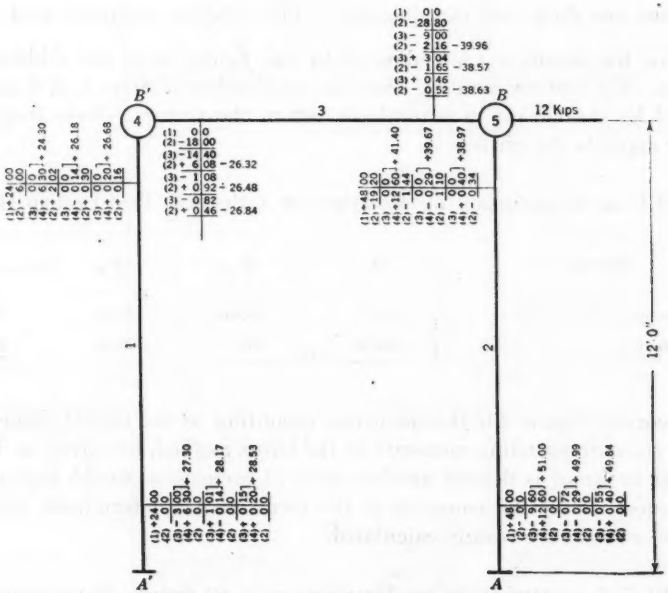

FIG. 31.—MOMENTS IN A TWO-LEGGED BENT.

1.—Calculate the moments in the columns due to the lateral forces, considering the joints fixed against rotation, but free to deflect laterally. The sum of the moments at the top and bottom of all the columns of a story is equal to the shear in the story, multiplied by the story height, and, as the deflections of the columns in a story due to the lateral forces are equal, the column moments and shears are proportional to the $\frac{I}{L^2}$ values of the columns. (When all the columns of a story are of equal height the values of I, or $\frac{I}{L}$ may be used in proportioning the moments.)

2.—Distribute the moments at the joints, considering them free to rotate, but not changing their location. (Same process as for vertical loads.)

[8] This is the convention suggested by L. E. Grinter and A. H. Finlay, Assoc. Members, Am. Soc. C. E., in their discussions of this paper.

3.—Carry over the distributed moments using a carry-over factor of $+\frac{1}{2}$ as is done for vertical loads.

4.—Balance the column moments in each story by making their sum equal the shear in the story times the story height. This assumes the joints to be fixed against rotation, but free to deflect laterally as in the first step. This completes a cycle and the designer is now ready to repeat Steps 2, 3, and 4 as many times as the desired accuracy may require.

To illustrate the procedure, the calculations are given in Fig. 31 for a simple two-legged bent with the bottom of the columns fixed. The loads and dimensions are shown on the diagram. The relative magnitudes of the $\frac{I}{L}$ values for the members are indicated by the figures near the middle of the members. The factors resulting from the application of Steps 1, 2, 3, and 4 are indicated by the figures in parenthesis before the factors. Note that Step 4 does not apply to the girders.

TABLE 5.—COMPARISON OF MOMENTS IN A SIMPLE, TWO-LEGGED BENT

Methods	M'_A	M'_B	M_A	M_B
Elastic theory	28.71	27.00	49.72	38.57
Cross method	28.53	26.68	49.84	38.97

The correct figures for the moments, according to the elastic theory, compared to the corresponding moments by the Cross method, are given in Table 5. If greater accuracy is desired another cycle of corrections would require but a few minutes. After the moments in the members are determined, the shears and direct stresses are readily calculated.

TABLE 6.—COMPARISON OF MOMENTS IN A 20-STORY, SYMMETRICAL, THREE-SPAN BENT

Story	Column *A*				Column *B*				Girder *AB*				Girder *BB*	
	Top		Bottom		Top		Bottom		M_{AB}		M_{BA}			
	Slope-deflection method	Cross method	Slope-deflection method	Cross method	Slope-deflection method	Cross method	Slope-deflection method	Cross method	Slope-deflection method	Cross method	Slope-deflection method	Cross method	Slope-deflection method	Cross method
1	15.02	15.39	22.84	22.75	21.08	21.06	25.88	25.59	23.98	23.32	20.43	19.70	20.56	19.98
2	9.41	10.19	8.96	9.06	18.87	18.67	19.91	19.18	16.99	15.70	15.30	14.10	18.80	17.36
3	8.35	9.01	7.58	8.06	15.67	15.17	15.24	14.58	15.68	14.22	14.32	12.83	15.83	14.31
4	7.33	7.91	7.33	7.94	14.68	14.08	14.48	13.95	14.20	12.89	12.80	11.52	15.37	13.90

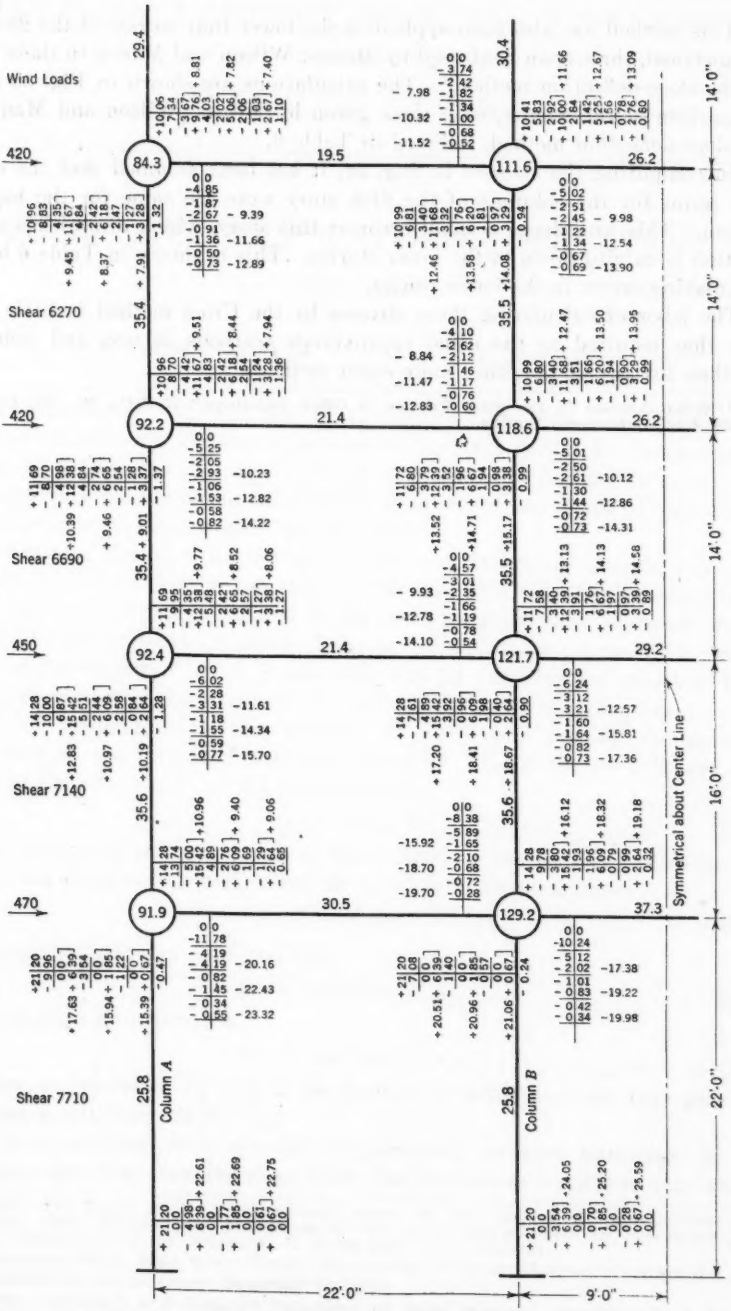

FIG. 32.—SYMMETRICAL THREE-SPAN 20-STORY BENT.

This method has also been applied to the lower four stories of the 20-story, symmetrical, three-span bent used by Messrs. Wilson and Maney in their paper on the slope-deflection method.[9] The calculations are shown in Fig. 32 and a comparison of the results, with those given by Messrs. Wilson and Maney by the slope-deflection method, is listed in Table 6.

In computing the stresses in Fig. 32, it has been assumed that the carry-over terms for the columns of the fifth story were the same for the top and bottom. This introduces a small error at this story, which diminishes as the solution is carried down to the lower stories. This is shown in Table 6 by the diminishing errors in the lower stories.

The labor of calculating these stresses by the Cross method is little more than that required by the usual approximate methods in use, and infinitely less than for any of the other more exact methods.

[9] "Wind Stresses in the Steel Frames of Office Buildings," *Bulletin No. 80*, Univ. of Illinois, Eng. Experiment Station.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

SOME ASPECTS OF WATER CONSERVATION

Discussion

BY MESSRS. W. H. R. NIMMO, AND C. R. OLBERG

W. H. R. NIMMO,[3] M. AM. SOC. C. E. (by letter).[3a]—The part of this paper dealing with reservoir capacities is based on the fact (which has been demonstrated by Hillman[4]) that the capacity of a reservoir at any level may be expressed very closely as an exponential function of the depth of water, and it is of interest to inquire why this should be so.

The size of the particles which a stream can transport depends to a considerable extent on their shape and the material of which they are composed, but if they were all spherical and of the same material, their diameter would vary as the square of the velocity of the stream and their volume as the sixth power of the velocity.[5] If it is assumed that all particles are spherical then the force exerted by the stream on a stationary particle is proportional to the change of momentum in that portion of the stream passing the particle and varies as the square of the mean diameter, D_p, of the particle and the square of the velocity, V, of the water, that is,

$$F = k\, D_p^2\, V^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

The frictional resistance due to the weight of the particle varies as the cube of the diameter so that, when the particle is about to move,

$$F = f\, D_p^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Equating Formulas (30) and (31),

$$V^2 = \text{constant} \times D_p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

but from the Chezy formula,

$$V^2 = \text{constant} \times r\, s \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

therefore, $r\,s$ varies as D_p and, if the hydraulic radius does not vary greatly, the slope, s, will vary as D_p.

If it is assumed that the size of particles increases uniformly in an up-stream direction, then the slope must also increase at a uniform rate which

NOTE.—The paper by R. A. Sutherland, Assoc. M. Am. Soc. C. E., was published in September, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: January, 1931, by Messrs. E. C. Eaton and A. W. Simonds.

[3] Designing Engr., Main Roads Comm., Queensland, Brisbane, Queensland, Australia.

[3a] Received by the Secretary, December 10, 1930.

[4] "The Capacities and Capacity Equations of Some Western Australian Dam Sites," *Transactions*, Inst. of Eng., Australia, Vol. II, 1921.

[5] "Physical Geography," by P. Lake, Cambridge Univ. Press, 1925.

is a property of a parabola. The foregoing appears to provide a physical explanation for the observed fact that the longitudinal section of a stream tends to be parabolic throughout most of its length.

Considering the case of the denudation of a slightly sloping block of land, such as a plain ($k\,m$) uplifted above the sea, as represented in Fig. 23, it will be noted that, at first, there can be no permanent stream, and denudation will take place only by weathering and the intermittent removal of material when rain falls. Gradually, run-off from storms will collect into runnels and, finally, into consequent streams running in the direction of the slope. Once formed, these streams will increase in volume and velocity as they proceed. Therefore, their erosive power will increase also, and they will steepen their beds to a convex curve such as $a\,b$ (Fig. 23). The stream will continue to erode until a narrow valley has been cut back to the position, $c\,d$; and the process will tend to continue to the position, $e\,f$ (Fig. 23), etc., back to the water-shed.

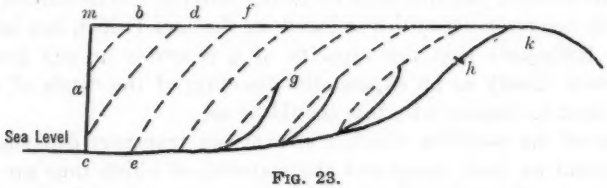

FIG. 23.

Erosion, however, cannot take place below the sea level at c and, having reached this point, the stream will commence to deposit material. At any point, such as e, erosion cannot take place below a level determined by the slope necessary to maintain a velocity that will transport the load which the stream is carrying. Consequently, a concave curve, such as $C\,e\,g$, will now be superimposed on the previous convex curve and the line of the valley will be eroded in gradual stages back to the position, $c\,e\,h\,k$, the portion, $c\,e\,h$, being concave and parabolic and the portion from h to the water-shed at k being convex. The term, concave, is used relative to the surface of the earth, which is considered to be flat over a limited area. Usually, the bed of a river will not be concave to the sky. The concave curve cannot proceed beyond the actual source of the stream, which will usually be a spring at about the point, h, since above this point there will not be sufficient accumulation of groundwater to maintain a flow during the interval between storms. Therefore, denudation here will proceed by weathering and the water-shed will usually remain rounded.[6]

A young stream which is eroding the bed of a narrow gorge near the head of its drainage area will usually be fully loaded with detritus. It will not also be eroding the sides of the valley which will be of a convex shape. The shape of a storage basin in such a locality will be similar to that shown in Fig. 24. Adopting the author's notation as far as possible, the area of any typical cross-section, such as $b\,c\,d$, is given by,

$$a = \int l\,dh = \int D\,h^v\,dh = \frac{D}{v}\,h^{v+1} = C_A\,h^n \ldots\ldots\ldots\ldots (34)$$

[6] See, also, "The Curve of Stream Erosion," by John Challinor, *The Geological Magazine*, Lond., Vol. LXVII, No. 2, February, 1930.

and the capacity of the reservoir will be,

$$w = \int a\, d\, l' = \frac{D}{v} \int h^{v+1}\, d\,(P\, h^z) = \frac{D\, P}{v} \int h^{v+z}\, dh = \frac{D\, P}{v\,(v+z)}\, h^{v+z+1}$$

$$w = C\, w\, h^m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

Equations (34) and (35) are identical with the Equations (1) and (2).

As the author points out, the value of n for a convex valley, such as Fig. 24, will be greater than 2 and, therefore, the value of v will be greater than unity. Since dams are usually built at control points such as A, B, or C, of Fig. 2, the longitudinal section of the stream may be considered to be parabolic and the value of z, therefore, will be 2. Hence, the value of $m = v + z + 1$ will be greater than 4; values of m greater than 4.5 do not seem to occur. In a previous discussion[7] of an abstract of the author's paper, the writer suggested the classification of reservoirs into gorge, hill, foothill, flood-plain, and lake types according to their value of m. The author has developed the idea in rather more detail, combining the flood-plain and foothill classes into one type, and adopting a definite range of values of m for each type, and his classification appears to be satisfactory.

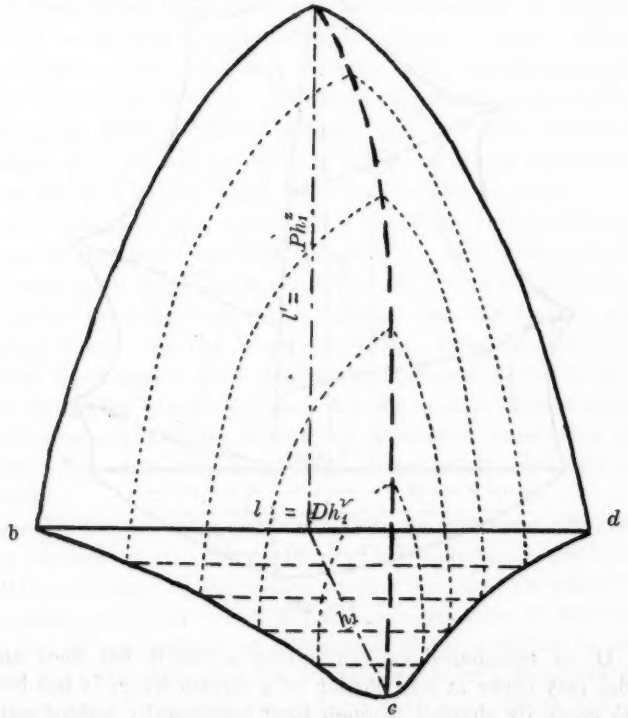

FIG. 24.

If a reservoir is located some distance from the water-shed, but in hilly country, there will be a larger flow of water to transport eroded material than

[7] *The Commonwealth Engineer*, Vol. 15, No. 11, June 1, 1928.

in the foregoing example and the stream will have cut lateral valleys of parabolic section. Generally, the lower portion of the sides of the main valley will have become parabolic and concave although the actual water-shed will still be convex, so that the value of v will be 2. The longitudinal section of the main stream will now be on a flattened portion of the parabola so that z will be nearly unity. Such a reservoir is illustrated in Fig. 25 and its value of m would be about 4. Following down the main stream still farther, the lateral valleys will also become flattened, and the valley will be V-shaped with $v = 1$, $z = 1$, and $m = 3$, corresponding to the author's hill type. Still farther down stream the grade of the main stream may be so slight that z approaches zero while v may still be nearly unity, and $m = 2$, corresponding to the author's flood-plain and foothill type. For the lake type the floor is approximately flat in all directions and for large shallow lakes both v and z will be nearly zero, so that m will be rather greater than unity, but it can never be less than unity. The lowest value of m thus far encountered by the writer was 1.04 for the Great Lake, in Tasmania.

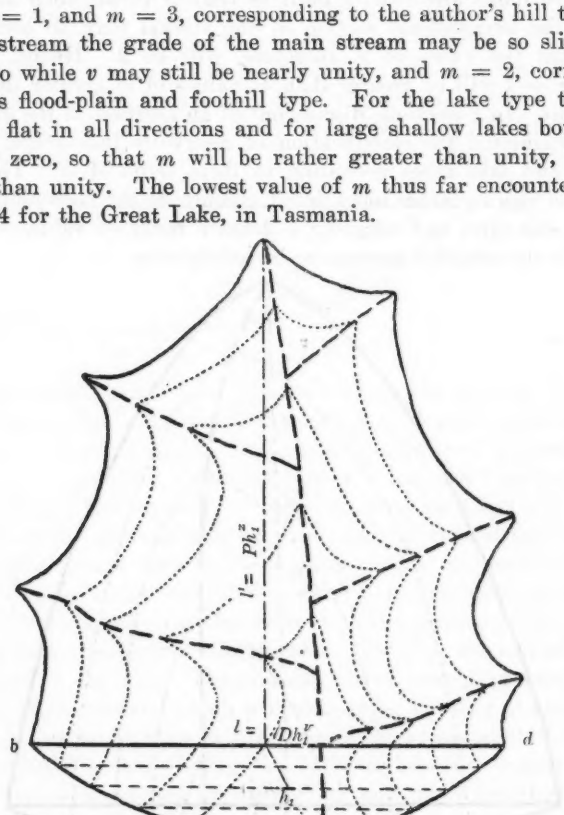

FIG. 25.

A true U- or box-shaped canyon having a nearly flat floor and nearly vertical sides may occur at any portion of a stream where it has been forced by uplift to erode its channel through hard horizontally bedded sedimentary rocks. In such a case, the value of v will be small and that of z may approach 2, so that m will probably be between 3 and 3.5.

The writer suggests the addition to the classification of a compound type to cover such reservoirs as Lake Cushman, Washington, which for the down-

stream half of its length consists of the narrow gorge of the Skokomish River and for the up-stream half of the expanse of Lake Cushman itself. For the lower portion of the capacity, water is stored in the gorge only and $m = 4.09$. As the water level rises, a greater percentage of the storage is in the lake and the value of m decreases gradually until for the upper 100 ft. its value is only 1.4. The capacity line on logarithmic paper is a curve, whereas for all the other reservoirs investigated by Mr. Sutherland or the writer it is a straight line.

From Equation (35) it is evident that as the values of v and z decrease (they usually do in a down-stream direction), the value of C_w must increase and this is clearly indicated in Table 1.

The application of the exponential formula may save much expenditure on investigational surveys since the capacity of a storage can usually be determined with sufficient accuracy from areas at two or three levels only.

Logarithmic site-area graphs plotted by the writer for a number of actual dam sites confirm the results obtained by the author and show that the area of any site to any given height can be represented closely by an exponential equation. The writer has also plotted widths against heights on logarithmic paper for a number of dam sites and, in many cases, the plotted points depart considerably from a straight line. Evidently, Equation (7) holds less closely than Equation (1) from which it is derived and, since the author's formulas for the volume of dams are based upon Equation (7) it might be expected that they would yield results which were considerably in error.

The writer computed the volume of a number of existing or proposed gravity dams by means of the formula given by Mr. Sutherland and compared the result with the volume calculated by the usual methods. It was found that, for sites which were fairly symmetrical, the formula gave quantities agreeing closely with the actual quantities. Even for decidedly unsymmetrical sites the formula gave results which were sufficiently accurate for preliminary estimates, since the error did not exceed that usually present owing to uncertainty as to the amount of foundation excavation which will be necessary. For earth or rock-fill dams, the formulas will also give satisfactory results.

The formulas for the volume of constant-angle arch dams (Equation (26)) involve the assumption that the contours of the foundation are both straight and parallel to the axis of the valley, or that the line, on which the cross-section is taken and from which the site-area equation is derived, is such that the effect of convergence or divergence of the contours is neutralized. Some of the most favorable sites for constant-angle arch dams exhibit considerable convergence of the contours and for such sites errors exceeding 30% in the computed volume of the dam may occur if the site-area equation is determined from a single cross-section taken along a chord. The composite section which is now proposed by the author will usually eliminate most of the convergence error.

The factor, $\frac{\phi}{2 \sin \frac{\phi}{2}}$, represents the ratio of the arc to the chord. The writer computed the ratio at the crest of fourteen dams and found that the value varied from 1.40 for the Pathfinder Dam to 1.015 for the Upper Otay Dam. He concluded that a constant factor of 1.05 would not be suitable for all dams. The factors given by the author in Table 3 vary with the value of n and the value of the crest angle, and their introduction will eliminate much of the error due to the use of a constant factor.

The author's equations also involve the assumptions that the radius varies uniformly from crest to base and that the subtended angle and the cylinder stress are constant from crest to base. For the Warragamba, Pacoima, King River, and Bullards Bar Dams, the writer has found by plotting curves that the value of the radius does vary almost uniformly from crest to base, and that the subtended angle varies greatly near the base. In high dams it is only approximately constant for the upper half of the dam. For the Warragamba and Pacoima Dams, the sites of which are fairly symmetrical, the cylinder stress is approximately constant at all levels; but for the Bullard's Bar, Salmon Creek, King River, and Lake Cushman Dams, it varies by as much as 50 per cent.

The site of the proposed Warragamba Dam is a straight gorge cut in sandstone, and the contours are practically straight lines parallel to the river. The computed volume of 261 000 cu. yd., given in Table 2, is for a preliminary design of a constant-angle arch dam made some years ago by the writer for this site, by means of the cylinder formula. The site is unusually symmetrical, and the probable depth of foundation excavation was allowed for in determining the site-area equation. The radii vary uniformly from crest to base, and the cylinder stress is unusually constant throughout. The actual volume calculated by ordinary methods is 284 000 cu. yd., so that the result given by the formula is 8% too small. The subtended angle is not constant, and the site-area equation was obtained from a section along a chord, instead of along the arc, and the error must arise from these two causes since all the other conditions required by the formula were satisfied.

Notwithstanding the various assumptions on which the author's formula for the volume of constant-angle arch dams is based, it appears from practical applications that in most cases the formula will give a quantity that is sufficiently near the true volume to be useful in making preliminary investigations, but the results may be expected to be usually less accurate than those obtained by the formula for the volume of gravity dams.

With the exception of the Gibson Dam, the actual volume of which is not shown, Table 2 does not include quantities for any dams designed by the trial-load or similar methods, or for any of the recently proposed high dams in which high compressive stresses are adopted. There is, therefore, no evidence to show what error would be involved in computing the volume of such dams by the author's formulas. The volume of the earlier arch dams, designed before the publication of the Cain formulas, are now of little practical interest.

A V-shaped valley, for which $n = 2$, is the most favorable for a constant-angle arch since the subtended angle can be kept approximately constant. For a valley with flat floor and vertical sides, $n = 1$, and a constant-angle arch becomes identical with a constant-radius arch.

For any valley with sloping sides (no matter how steeply), the constant-angle type of arch will contain less material than the constant-radius type. The comparison of specific capacities for different purposes in Figs. 17 and 18 is very instructive.

The writer has read this paper with much interest, and its value for practical use is much enhanced by the addition of the alignment charts.

C. R. OLBERG,[8] M. AM. SOC. C. E. (by letter).[8a]—The author has performed a distinct service to the profession in calling attention to the use of logarithms as providing a means of comparison between reservoir sites and dam sites, respectively. As Mr. Sutherland advises, the quantities derived from his methods cannot be used as a basis for final estimate, but they promise to be invaluable for the purpose of preliminary estimates and they also afford a method for direct comparison between various dam sites, and between different types of dams of various heights of the same site, with a minimum of work.

It is a definite achievement, since heretofore this was not possible, without days or even weeks of arduous labor, as those who have to pass on the relative excellence of proposed dam sites, or the respective merits of various types of dam at the same site, are well aware.

Most of the items considered are clearly and ably presented, but the author himself confesses that through the necessity of reducing the length of his original paper, the discussion of some of the subjects is somewhat "sketchy". It is to be regretted that Mr. Sutherland felt constrained to limit his discussion on "economic installation" to references to previous publications, which unfortunately are not readily available.

Reservoir Sites.—It has long been observed that capacity and area curves of reservoirs, follow some form of exponential curve, but it has remained for the author to demonstrate this fact in the application of logarithms to determine the equation.

The general form of an exponential curve in the nomenclature of the author (see Equation (2)) is $w = C_w h^m$.

Passing to logarithms,

$$\log w = \log C_w + m \log h \ldots\ldots\ldots\ldots\ldots\ldots (36)$$

Obviously, this is the equation of a straight line, in which, m is the tangent of the slope and $\log C_w$, the intercept on the y-axis.

Regarding a reservoir, if the logarithms of corresponding depths and capacities are plotted on co-ordinate paper with equal horizontal and vertical scales, it will be found that they will be nearly on a straight line. The straight line that best satisfies the points is the logarithmic equation, from which the algebraic formula, Equation (2), may readily be written.

[8] Cons. Irrig. Engr. to the Govt. of Transcaucasia, Tiflis, Union of the Socialistic Soviet Republics.

[8a] Received by the Secretary, December 23, 1930.

Since two points are required to determine a straight line, the capacities corresponding to two depths are all that is necessary to obtain the approximate capacity of a reservoir at any other depth. However, three or more points will permit a better adjustment of the theoretical to the actual curve. As pointed out by the author, it is remarkable how close the theoretical agrees with the actual capacity.

The general equation can readily be used to extend the capacity curve of a reservoir beyond the limit of the original survey, a necessity that sometimes arises. The first derivative of the general equation with respect to h_1 is, $dw = C_w\ m\ h_1^{m-1}\ dh_1$, or, $s = C_w\ m\ h_1^{m-1}$, which is Equation (4) in Mr. Sutherland's paper. From Equation (4) the area of the reservoir, s, at any depth, h_1, may be computed.

The corresponding logarithmic equation is,

$$\log a = \log C_w + \log m - (m - 1) \log h_1 \ldots\ldots\ldots\ldots (37)$$

Equation (37) is also a straight line for which $m + 1$ is the tangent of the slope and $\log C_w + \log m$, the intercept on the y-axis. Equations (4) and (37) may be of assistance in problems of evaporation and seepage losses and, also, in extending the area curve.

The author's classification of reservoirs into types in accordance with the value of m is interesting, but in the writer's opinion this is important only as a means of definition and comparison.

It is not clear what particular means of mathematical legerdemain was used to demonstrate that m is the ratio of the maximum depth to the average depth of the reservoir. The writer must also confess ignorance of the real meaning of the constant, C_w, except that h_1^m may be regarded as a unit of measurement for each particular type of reservoir, and C_w, a scalar number denoting the number of times the quantity is taken as a unit.

The preparation and use of line charts is an ingenious method of determining quickly the values of C_w and m, but the writer believes that there is an advantage in plotting the logarithmic line, in order to permit an adjustment of the theoretical to the actual curve. Possibly the same results may be secured by adjusting the points within the polygon formed by three or more lines on the line chart somewhat analogous to a solution of simple three-point problems in plane-table work.

Dam Sites.—Mr. Sutherland points out that the theoretical area of a dam site may be found in a manner similar to that for determining the theoretical capacity and area curves of a reservoir. To the writer the simplest method of procedure would be to plot the logarithm of the width of the dam site at critical elevations against logarithms of the corresponding depths.

The tangent of the slope of the straight line adjusted through the points will be the exponent, p, of the independent variable, h, and the intercept on the y-axis will be the coefficient, B, in the general equation,

$$w = B\ h^p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

in which, w is the theoretical width of the dam site at any height, h. Equation (38) is the trace, on a vertical plane, of the theoretical dam site.

Integrating Equation (38) with respect to h,

$$A = p\ B\ h^{p-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

which is the general area equation.

In Equation (38) letting w equal l, $B = n\,C_a$ and $p = n - 1$; then $l = n\,C_a\,h^{n-1}$, which appears to be derived by the author directly from the capacity equation for the reservoir (see Equation (7)). Until the relation between reservoir and dam site is more definitely established, the writer feels that this is putting assumption to rather a severe test.

Dams.—Equation (2) is proposed by the author for determining the volume of a triangular dam. Substituting for t and l in this equation, their equivalents, $Kg\ (H - h)$ and $n\ C_a\ h^{n-1}$, respectively, and integrating with respect to n, he derives Equation (10) as an expression for the volume of the dam. Substituting for n and C_a in this equation, their equivalents, $p + 1$ and $\frac{p^B}{p+1}$, respectively, results in the formula,

$$v_g = \frac{B\ K_g\ H^{p+2}}{(p+1)\ (p+2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

in which, the exponent, p, and the coefficient, B, are derived from Equation (39), the integration of the theoretical curve of the dam site. The same result could have been obtained by integrating Equation (39).

If suitable allowances are made for the concrete in the foundations along the base and abutments, and an appropriate quantity is added for crest width, Equation (40) will give a close approximation of the volume in a gravity dam, at the site under consideration.

The author has done a large amount of work and displayed considerable ingenuity in developing approximate formula for arch and constant angle dams. These two types, with the variable radius dam, are more dependent than others on the shape of the dam site for their stability and volumetric contents, and for this reason cannot be compared directly with a gravity dam.

The proposed formula and the presentation in alignment charts will be of great assistance since it renders this comparison possible, without the labor required to design a series of curved dams for each site. The error involved in the use of the cylinder formula, however, may be considerable, and the writer suggests the possibility of applying a factor to the volume as determined by the cylinder formula that will reduce, if not eliminate, the error.

The volumes of other types of dams, including the nail-head, multiple-arch, multiple-dome, and Ambursen, can be compared almost directly to the volume of a gravity dam of equal height, since they are only slightly affected by the shape of the dam site. It is probable that some of the factors will vary as a power of the height, but the relation should approximate a straight line.

Specific Capacity.—The writer believes that a more accurate determination of the "specific capacity", as defined by the author, could be secured by making an adjustment, providing for the clearance of the crest of the dam above the reservoir surface and the depths of the foundations below the bottom of the reservoir, so that the variable, h, of the reservoir would correspond to

the variable, h, of the dam. It would complicate the equations somewhat, but it is not believed the difficulty would be insurmountable.

While the specific capacity will afford a ready means of comparing dams of variable heights of the same type, the reciprocal of the specific capacity (or the number of cubic units of the dam per unit of storage capacity) would afford a better means of comparison between dams of different types.

Costs.—A factor, as suggested by Mr. Sutherland, applied to the various materials will then give a direct comparison of cost between the various types of dams. This information could be plotted on a logarithmic chart, on which the variation in costs of the several types of dams of varying heights, for the same site, could be clearly and concisely shown.

The cost of the accessories of a dam, including spillways, outlet gates, etc., depends directly upon the type selected, and it is evident that the size, which influences the cost, is more a function of the reservoir than the dam. For dams of the same height of different types, the capacities of the spillways and outlet gates will be the same, and if the same type of accessories is used, the cost will be a constant. For different heights of dams of the same type, the capacity of the spillway will vary as some negative power of the depth of reservoir, and the capacity of the outlet gates will remain practically constant.

On account of these varying conditions, it appears to be hopeless to apply a factor to the cost of a dam to determine the cost of the accessories, except as the roughest kind of an approximation. These costs, like the cost of rights of way, must be computed separately and added to the cost of the dam, to give the total cost of the project.

Conclusion.—The form in which the author presents his equations dispels any lingering hope the writer may have had respecting a maximum specific capacity. It was a fond hope that had been long entertained. The economic height must be sought in other relations than those which exist between the height of a dam and storage capacity. The loss in this respect, however, is more than compensated for by the assistance which the data, so ably presented by the author, has been to the work at hand. It must be recognized that the results are approximate, but in an instance under observation they have been close approximations and serve as a firm basis, on which to form an opinion respecting the relative merits of various heights and types of dams for the same site, and to form a means of comparison between various dams and reservoir sites.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

CONSTRUCTION OF LA OLA PIPE LINE, IN CHILE

Discussion

BY MESSRS. ERNEST B. NELSON, H. L. THACKWELL, AND WILLIAM E. RUDOLPH

ERNEST B. NELSON,[1] ASSOC. M. AM. SOC. C. E. (by letter).[1a]—The author is to be commended for this very interesting description of La Ola Pipe Line and especially for the valuable unit construction costs presented. These low unit costs indicate a very efficient organization, as is likewise evident from the rapid progress of the work which was carried out in the face of many adverse and difficult conditions far from the source of supplies. As is the case on many construction jobs, transportation proved to be a major problem in maintaining progress to the required schedule.

The original design for this pipe line (1917-18) called for continuous wood-stave pipe for the low-pressure portions, about 80% of the total length, with lap-welded steel pipe at points of high pressure, including of course the Pasto Cerrado (now Montandon) Siphon. Conditions at that time with the World War in progress, made it necessary to reduce to a minimum the use of steel in all units of the projected plant development, since deliveries of steel plate, structural shapes, etc., were very slow and uncertain. During this period, due to the fact that steel construction had to be virtually abandoned, the last 20 km. of the railroad from Pueblo Hundido to the plant at Potrerillos—a difficult section in the mountains—was relocated in order to eliminate several proposed steel bridges. Four additional tunnels were thus introduced, making a total of eleven in this portion of the line, but the work was continued without serious interruption so that the railroad was completed and placed in operation early in 1919, a few months after the signing of the Armistice.

Work on the tunnels for the pipe line had been discontinued. Later, it was resumed, another tunnel having been introduced in the meantime by an alternate location through Pedernales Ridge. Late in 1920, with six tunnels completely driven and the Pedernales Tunnel 60% through, a general shutdown of construction put a stop to further activities.

NOTE.—The paper by W. B. Saunders, M. Am. Soc. C. E., was published in September, 1930, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

[1] Supt. of Concrete Constr., Anglo-Chilean Consolidated Nitrate Corp., Tocopilla, Chile.

[1a] Received by the Secretary, November 29, 1930.

When the work was opened up again early in 1925, the change to steel pipe throughout had been adopted, and the project was carried through to completion as described by the author. Steel pipe besides being more dependable and less likely to give trouble in operation than either wood-stave or concrete pipe, has the further advantage of a minimum loss or waste of water. Everywhere in a desert region, such as Northern Chile, water is scarce and inadequate, and in this case the entire flow of the Ola River is utilized during half the year, or more.

For a wood-stave pipe line of the size and length here considered, leakage, including seepage and evaporation losses due to saturation of the staves, may reasonably be taken at about 4% of the total flow, or well over 1 cu. ft. per sec., which would have meant just that much less water available for use at the plant. It is apparent that for conditions in which the supply of water is strictly limited and possibly insufficient for the demands, this item of leakage may be a determining factor in the selection of the type of pipe to be used.

The author may have some data from observations of the carrying capacity of this line, which would be of general interest for comparison with some of the accepted formulas. If the writer remembers correctly, at several points in the line, provision was made for measuring the flow by means of a sharp-crested weir in a short section of open canal.

H. L. THACKWELL,[2] ASSOC. M. AM. SOC. C. E. (by letter).[2a]—The local conditions, trials, and labors of constructing a bold and daring water-supply project in the high pampas of Northern Chile, has been ably presented in this paper.

The casual reader can better appreciate this work by comparing it with a similar undertaking, such as crossing the Grand Canyon of Arizona with a pipe line, except that in Chile the "quebrados", or canyons, do not have running streams.

The transportation problem is particularly difficult to solve. All foodstuffs must be imported due to the arid climate, which is unsuitable for agriculture. This very lack of vegetables has made the Northern Chilean a meat eater, and largely a protein feeder.

As stated under the heading, "Water Supply", the "atmosphere is absolutely dry"; in fact, it is so much so that the mucus membrane in one's nostrils dries up, and will crack and bleed unless greased with vaseline or similar ointment. The statement that "the air is brilliantly clear" can be better understood by the study of photographs taken of mountains 25 and 40 miles away, which clearly show geographical demarcations and variations in texture and conformity. The only fuel is the root of a small "wind-blown" bush somewhat similar to the sagebush found in the Western United States.

The wind not only "reaches a velocity of 20 miles per hour * * * every day", but it carries sand and small pebbles which cut like emery. Facing a high desert wind on the La Ola Pampa is veritable torture; even burros turn their backs and seek shelter. Engineers' stakes have been eroded by the pampa

[2] Cons. and Constr. Engr., Jacksonville, Tex.

[2a] Received by the Secretary, December 22, 1930.

winds to a depth of 0.25 in. in a year's time, erasing all marks of identification.

Chilean laborers are strong and capable and, in many ways, remind one of the Japanese; they are short, stocky, and able to carry large loads easily without tiring. The "rotos" are a distinct class of labor, and when recruited from the desert districts they become efficient workmen. Chilean miners working by hand in the tunnel headings, make as good progress as the average Western miner of the United States, using the same tools and methods.

The water supply from the Rio Ola is fairly constant as can be seen from Fig. 4. It is quite possible to obtain 29 cu. ft. per sec., provided a storage of 4 000 acre-ft., is available. A study of Fig. 13, will show that a maximum possible draft is available with adequate storage, and that if the pipe line is designed for 26 sec-ft., there will be periods every year when the mill will have to reclaim several second-feet of water. A mean draft of 22.5 cu. ft. per sec. is practical with a storage of 100 acre-ft. at the mill site, near Potrerillos. In Fig. 13, the hatched area below 26 cu. ft. per sec. indicates used water that should be reclaimed at the mill. Water flow will fall below the requirements about one-half the time.

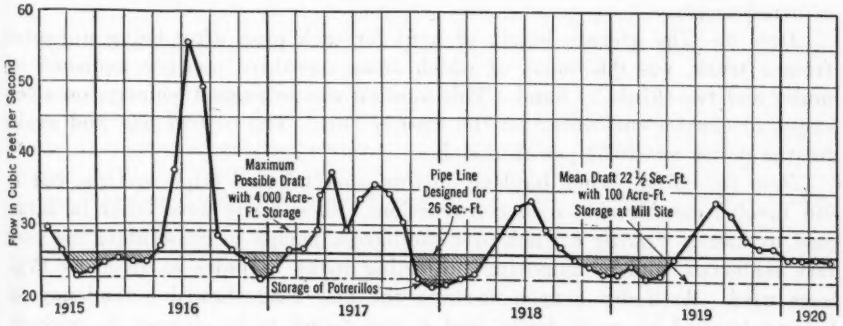

FIG. 13.—HYDROGRAPH OF RIO OLA.

The author is to be congratulated for the excellent manner in which the final construction work was completed, and, in particular, he should be commended for the difficult organization work and effective training of inexperienced labor and the economical results obtained by skillful supervision.

WILLIAM E. RUDOLPH,[3] M. AM. SOC. C. E. (by letter).[3a]—Some costs compiled by the writer might be of interest in comparison with those tabulated by Mr. Saunders. This particular undertaking was not a new pipe line, but concerned the construction of looping lines to increase the capacity of an existing water supply line.

The line in question was 55 miles long, 47 miles of which was 9-in. pipe and the remainder, 6-in., with all screw couplings. Its capacity had been impaired about 30% by nodular incrustations, and at one section the hydraulic grade had fallen below the level of the pipe. The measures taken to regain a

[3] Chf. Engr., Green-Muth Bldg. Corporation, Rockville Center, N. Y.

[3a] Received by the Secretary, December 23, 1930.

portion of the lost capacity consisted of substituting a section of new 9-in. pipe for the 6-in. pipe at one location and a section of new 12-in. pipe for the 9-in. pipe at another. In each case the pipe that was removed was relaid beside the existing line in the adjoining section, this division of the flow between two parallel lines further reducing the flow friction losses.

Unit costs on this work are shown in Table 5. The following notes are in explanation of certain units in the tabulation:

Item 1.—The cost of housing and feeding a laborer in camp was about equal to the average wage paid. As this expense was considered part of the general overhead, it has not been charged against the various units. However, it can be distributed by doubling the units for labor under each of Items 3 to 10, inclusive.

Item 2.—The unit cost of pipe would appear high as compared to costs given by Mr. Saunders. This was partly due to the large number of specials required. On work of greater magnitude, and on which larger sizes of pipe were used, it would undoubtedly have been advantageous to assemble the pipe in the field as described by Mr. Saunders.

Item 5.—The trucks hauled average loads of 2 tons on outgoing trips and returned empty.

Item 6.—The average length of haul for each pipe, after being unloaded from a truck, was 0.65 mile, of which about one-third was accomplished by mules and two-thirds by hand. This hauling was in rugged country, on steep slopes of canyon enclosures for the greater part. The cost of hay and grain for the mules was $0.51 per animal day.

Item 10.—Motor-truck haulage on this work effected large savings, but it did involve considerable road construction. Such roads were build in large part by merely driving a truck over the barren pampa to consolidate the surface materials, and by removing projecting rocks. Grades as steep as 15% were used. At a dry canyon crossing the new road showed a tendency to become blocked by snow drifts, and it was found to be cheaper to relocate the road than to shovel snow from the earlier location.

High labor turnover was experienced. Little difficulty was encountered in recruiting men at the start, because although it was late April (the beginning of the winter season), Chilean workmen sought employment where lodging and board were free, even if it did involve living in tents at an elevation of 13 000 ft. In most cases, such enthusiasm lasted only until the first pay day. Later, the depleted ranks of workers were in part filled with native Bolivians, or more exactly, Indians, from the small farming centers of the mountain slopes. These "pueblos" exist upon the community system as in the days of the Inca, and they have little, if any, connection with the outside world. It was necessary to obtain permission from the head of each village to give employment to his men.

One would have expected that native Indians, accustomed to the altitude and the severe climate of the Andean highlands, would prove far superior to the average Chilean laborer. Such was not the case. Generations of inbreeding had worked its effects upon the Indians. The Chileans averaged

25% and more bonus on their contracts, the Indians scarcely earned their day's pay. As the job neared completion, it was the Chilean laborers who had

TABLE 5.—UNIT COSTS OF CONSTRUCTING "LOOPING" LINES FOR INCREASING CAPACITY OF TOCONCE PIPE LINE, SUPPLYING CHUQUICAMATA, CHILE, APRIL TO SEPTEMBER, 1927

No.	Item	UNIT COSTS, IN DOLLARS (8.18 PESOS)			
		Materials	Labor	Expenses	Total
1	Cost of maintaing men in camps per man-day:				
	(*a*) Camp supervision	.....	$ 0.03	.....	$ 0.03
	(*b*) Food and water	$ 0.62	0.11	$ 0.01	0.74
	(*c*) Fuel	0.03	0.01	0.01	0.05
	(*d*) Camp maintenance, including depreciation of equipment	0.14	0.06	.....	0.20
	(*e*) Transportation of camp equipment	.....	0.03	.....	0.03
	Total	$ 0.79	$ 0.24	$ 0.02	$ 1.05
2	Cost of new pipe, unloaded from cars per foot	$ 3.65	$ 0.04	$ 0.03	$ 3.72
	Cost of new pipe unloaded from cars per ton	158.95	1.75	1.30	162.00
3	Cost of uncovering old pipe per foot	.....	0.01	.....	0.01
4	Cost of taking up old pipe per foot	.....	0.03	.....	0.03
5	Cost of motor-truck haulage per ton-mile	0.18	0.02	0.04	0.24
6	Cost of haulage by mule and by hand per ton-mile	3.21	8.00	0.93	12.14
7	Cost of excavation per foot	0.02	0.14	.....	0.16
8	Cost of pipe-laying per foot	0.01	0.05	.....	0.06
9	Cost of covering pipe per foot	.....	0.06	.....	0.06
10	Cost of building road per mile	.....	381.25	.....	381.25
11	Engineering and supervision, percentage	.....	.....	.....	9

remained through devastating snow and wind storms, constructing pipe lines on exposed mountain slopes. The Indians had returned to the less rigorous life of repairing their irrigation ditches, down in the valleys.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TESTS OF BROAD-CRESTED WEIRS

Discussion

BY MESSRS. BORIS A. BAKHMETEFF, AND D. D. CURTIS

BORIS A. BAKHMETEFF,[1] M. AM. SOC. C. E. (by letter).[1a]—The remarks that follow are offered with the sole purpose of further elucidating the phenomenon described in Professor Woodburn's valuable paper.

"Theory" of Flow.—The author defines the critical depth as one "at which, for any given total head, the discharge is a maximum". It was Bélanger who, nearly a hundred years ago, first suggested the postulate of "maximum discharge" as a basis for the "theory" of flow over broad-crested weirs. Given the total head, $H = H' + \frac{v^2}{2g}$, and a crest sufficiently wide so that flow may become parallel (Fig. 17), the depth, d, is supposed to establish itself in such a manner that the discharge will be a maximum. This method of treatment has since been invariably followed by hydraulicians, notwithstanding the fact that the deductions from the "maximum discharge theory" do not hold in the face of experimental evidence.

In fact, designating the relation of the depth, d, to the head, H, as

$$K = \frac{d}{H} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and introducing a "velocity coefficient", ϕ, to take account of the hydraulic losses which occur between Sections A and B, the velocity, v. and the discharge, q, per unit width in Section B will be:

$$v = \phi \sqrt{2g(H-d)} = \phi \sqrt{2gH}\sqrt{1-\frac{d}{H}} = \phi \sqrt{1-K}\ \sqrt{2gH} \ldots (4)$$

and,

$$q = dv = KHv = \phi K \sqrt{1-K}\, H^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Designating the "discharge coefficients" in the usual weir formula by m and $M = m\sqrt{2g}$,

$$q = m\sqrt{2g}\, H^{\frac{3}{2}} = M H^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

NOTE.—The paper by James G. Woodburn, Assoc. M. Am. Soc. C. E., was published in September, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: October, 1930, by H. W. King, M. Am. Soc. C. E.; and December, 1930, by Messrs. Sherman M. Woodward and David L. Yarnell.

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[1a] Received by the Secretary, November 19, 1930.

by comparison with Equation (5),

$$m = \phi K \sqrt{1 - K} \quad \ldots\ldots (7)$$

and

$$M = \phi K \sqrt{1 - K}\sqrt{2g} \quad \ldots\ldots (8)$$

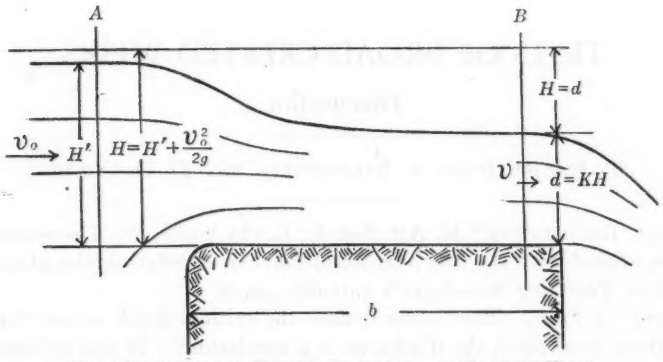

FIG. 17.—PARALLEL STREAM FILAMENTS IN UNIFORM FLOW.

Obviously, the "maximum discharge" is reached whenever m in Equations (6) and (7) is a maximum. For a given form of the entrance edge, ϕ may be assumed to be unchangeable. By writing

$$\frac{dm}{dk} = \phi \sqrt{1 - K} - \frac{1}{2} \frac{\phi K}{\sqrt{1 - K}} = 0$$

the maximum discharge is found to occur at $K = \frac{2}{3}$ irrespective of ϕ. In other words, if the maximum discharge were to govern, the depth, d, near the end of the crest would be $d = \frac{2}{3} H$ for any type of crest, irrespective of its length, roughness, or the form of the entrance edge.

This deduction is in contradiction to observed facts. Professor Woodburn's profiles (Fig. 4), and other experiments,[2] show K to be less than two-thirds. Moreover, the value of K varies with the conditions of flow. For rounded entrance edges, under average conditions, K is near 0.63. For sharp entrance edges and relatively long crest, the value of K is nearly 0.6 and eventually less.

Cognizant of the discrepancy between the traditional theory and experiment, the writer suggested some years ago that investigators should approach the problem from a different angle. "Energetic" considerations were to be applied. For a given total head, H, the depth, d, and the resulting discharge were assumed to be such as to make the contents of energy in Section B the smallest possible minimum.[3]

[2] See, for example, *Water Supply Paper 200*, U. S. Geological Survey.

[3] "On Varied Flow of Liquid in Open Channels," (in Russian), St. Petersburg, 1912; reprinted, 1928; also, "Hydraulics," Lectures at the Polytechnical Inst., St. Petersburg, 1913.

If a discharge, Q, flows with a uniform depth and with parallel stream filaments in a canal of given form, such as Fig. 18, and if the depth of flow, d, varies, there is a certain "specific energy of flow" that corresponds to each depth of flow:

$$\varepsilon = d + \frac{v^2}{2g} = d + \frac{Q^2}{2g\,a^2} \qquad (9)$$

which is the energy head (energy per unit weight) referred to the bottom line of the cross-section. The factor, a, in Equation (9) is the cross-sectional area at the depth, d. The graph of ϵ, which features the contents of specific energy as it changes under varying depth, has the shape illustrated in Fig. 19. The energy curve is tangent to the horizontal axis and to a straight line, drawn through O at 45%, and passes through a minimum point at C.

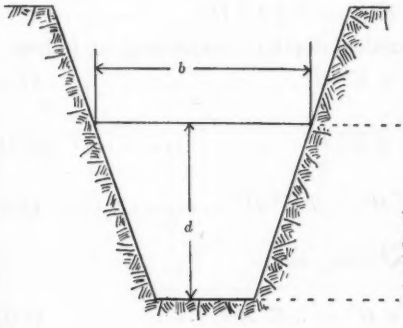

FIG. 18.

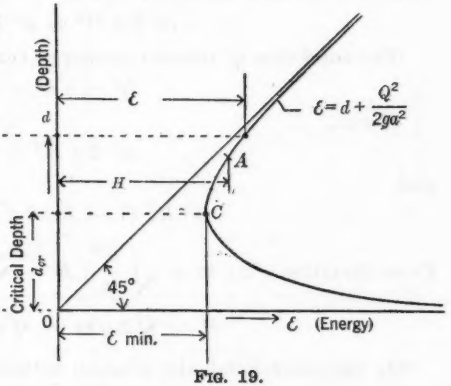

FIG. 19.

The ordinate, d_{cr}, at Point C is the critical depth. From this point of view critical flow acquires a definite physical meaning, namely, that at which the discharge in the canal flows with the smallest possible minimum content of energy per unit of weight. The energy curve, as shown in Fig. 19, and which has proved to be of incalculable service in considering different problems of varied flow, has been discovered and presented in different countries by different hydraulicians seemingly independent of each other. In the United States a lucid exposition has been given[4] by Julian Hinds, M. Am. Soc. C. E. In Germany, Professor Rehbock alludes[5] to a similar curve. In Russia, the energy curve was introduced by the writer[3] in 1912 and, as far as he is aware, that was its first appearance. For a canal of any form, the critical depth is determined by the equation, $\left(\frac{a^3}{b}\right)_{cr} = \frac{Q^2}{g}$, in which, a is the cross-sectional area, and b, the width of the surface at the depth, d_{cr}. For a rectangular canal, the critical depth is related to the discharge per unit width by the well-known formula used by Professor Woodburn, $d_{cr}^{\,3} = \frac{q^2}{g}$.

[4] *Engineering News-Record*, 1920.

[5] "Betrachtungen über Abfluss, Stau, und Walzenbildung," Berlin, 1917; see, also, Böss, No. 284, Forschungsarbeiten, V. D. I, 1927.

On the energy diagram, Fig. 19, flow over a broad-crested weir is shown by the movement from an initial point, A, corresponding to the energy content, H, down along the energy curve toward C. Obviously, in its natural tendency to become lower with a corresponding loss of energy, the level on the crest may not fall below Point C, as a further lowering of the depth could take place only through an increase, an addition of energy from outside, which is obviously impossible.

Thus, the critical depth (that at which flow takes place with a possible minimum content of energy), appears to be the natural limit, to which the surface level may lower, and which will establish itself at the end of the crest. The analytical relations between the different parameters of flow may be determined, as follows: The discharge in Section B, in accordance with Equation (5) and Equation (6), is:

$$q^2 = m^2\, 2\, g\, H^3 = \phi^2\, K\, (1 - K)\, 2\, g\, H^3 \ldots\ldots\ldots\ldots (10)$$

The condition of minimum energy (critical depth) is expressed, as follows:

$$q^2 = g\, d^3 = g\, K^3\, H^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Hence,

$$m^2\, 2\, g\, H^3 = g\, K^3\, H^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

and,

$$\phi^2\, K^2\, (1 - K)\, 2\, g\, H^3 = g\, K^3\, H^3 \ldots\ldots\ldots\ldots\ldots (13)$$

From Equation (12), $m = \sqrt{\dfrac{K^3}{2}}$; $K = \sqrt[3]{2\, m^2}$; and,

$$M = \sqrt{2\, g}\, m = \sqrt{g\, K^3} = 5.67\, K^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots (14)$$

On the other hand, the relation between K and the velocity coefficient may be expressed in the form:

$$\frac{2\, \phi^2\, (1 - K)}{K} = 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

which gives, $K = \dfrac{2\, \phi^2}{2\, \phi^2 + 1}$, and $\phi = \sqrt{\dfrac{K}{2\, (1 - K)}}$.

It should be noted that this analysis applies to crests that are horizontal or inclined at a slope not exceeding the "critical slope." For slopes beyond the critical, the very nature of the phenomenon changes, as outlined subsequently. Curves for various numerical values of Equations (14) and (15) are plotted in Fig. 20.

The factor, 3, is a "resistance coefficient", obtainable from the relation,

$$\frac{v^2}{2\, g}\, (1 + \zeta) = H - d \ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

which, in view of Equation (3), gives, $\zeta = \dfrac{1}{\phi^2} - 1$.

In contradistinction to the "maximum discharge" theory, K is no longer constant. In the "ideal" case, with $\phi = 1$ (no losses), K is two-thirds. Under actual conditions, with $\phi < 1$, K is less than two-thirds. The greater the losses the smaller the relative value of K. Preliminary experiments, carried

out during 1913 in the Hydraulical Laboratory of the Polytechnical Institute of St. Petersburg, gave results, which on the whole appeared consonant with the foregoing theory. The principal difficulty in comparing theoretical deductions with observations, lies in the fact that flow over a weir, unless the relative length of the crest, $\left(\frac{b}{H}\right)$, is considerable, is seldom "parallel" in the sense of the paragraph to follow. A large part of the surface is covered with waves. Moreover, the surface at the end of the weir is usually marked by pronounced curvature.

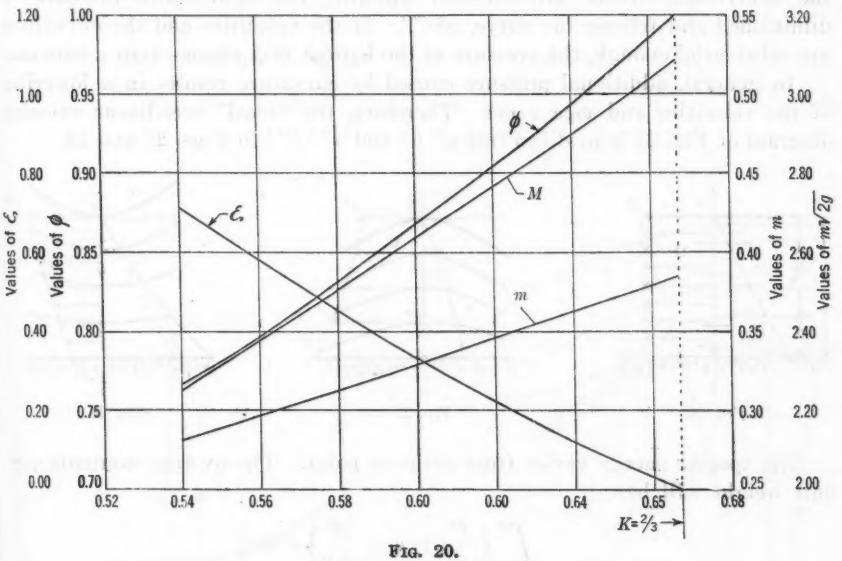

FIG. 20.

Effect of Curvature on the Critical Depth.—Professor Woodburn and Professor Webb both seek to determine the locus of the critical depth as a means of establishing a simple and reliable method of measuring the volume of flow. This locus is assumed by them to lie in a section, such that the depth of flow corresponds with the relation,

$$d'_{cr} = \sqrt[3]{\frac{q^2}{g}} \qquad (17)$$

in which, d'_{cr} is the "apparent" critical depth.

The main point to be taken into account in this connection is that the critical depth as defined by Equation (17) is determined under the assumption of parallel flow; that is, flow in which the stream filaments are practically parallel, having "no appreciable curvature nor divergence". These requirements for flow being "parallel", which underlie the whole theory of varied flow, were formulated by Bélanger as early as 1828. In this case, and only in such a case, the potential energy, $y + \frac{P}{\Delta}$, in the Bernoulli equation is identi-

cal for all and every point of the cross-section and is everywhere equal to the depth, d, which accounts for the simple expression of specific energy of flow as given in Equation (9). The physical characteristic of parallel flow is, that the pressures within a cross-section are distributed according to hydrostatic law, the pressure diagram being the hydrostatic triangle *abc* (Fig. 21).

Now, if the surface and the stream filaments are curved, the pressure diagram is no longer a triangle. Centrifugal forces come into action, which for concave filaments (Fig. 22) bear downward, thus adding to the hydrostatic pressure and changing the triangle into abc''. For convex filaments (Fig. 23), the centrifugal forces are directed upward; the hydrostatic pressure is diminished and follows the curve, abc''. If the velocities and the curvature are substantial enough, the pressure at the bottom may change into a vacuum.

In general, additional pressure caused by curvature results in a lowering of the velocities and *vice versa*. Therefore, the "ideal" rectilinear velocity diagram of Fig. 21 is modified into $a''\ b''$ and $a''\ b''$ in Figs. 22 and 23.

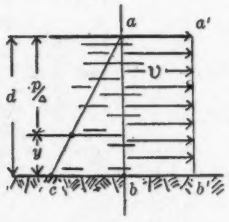

FIG. 21.

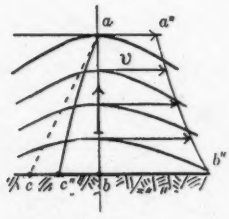

FIG. 22.

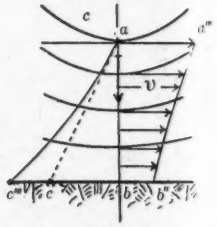

FIG. 23.

The specific energy varies from point to point. The average contents per unit weight will be:

$$\varepsilon = \frac{\int_0^d \left(\frac{P}{\Delta} + y + \frac{v^2}{2g}\right) v\, dy}{\int_0^d v\, dy} \qquad \text{.....(18)}$$

To determine the energy, one must know the distribution of pressures and velocities over a cross-section, a problem usually requiring recourse to methods of hydrodynamics.

To get a concrete appreciation of the effect of curvature, consider the elementary case of a canal, the bottom of which is laid on a circle of radius, R_0 (Fig. 24), under the further assumption that in the vicinity of the vertical section, *oab*, the filaments form concentric circles with a common center of curvature in 0; and, moreover, that the motion occurs in conformity with the law of the so-called "free vortex". In this case, as is well known, the velocity is inversely proportioned to the radii, so that $v\,R$ is a constant. In the case of concave streaming (Fig. 24 (*a*)), designating the surface velocity by v_a, the formula, $v\,(R_0 - y) = v_a\,(R_0 - d)$ is obtained, or,

$$v = v_a \frac{R_0 - d}{R_0 - y} \qquad \text{.....(19)}$$

The discharge is:

$$q = \int_0^d v\, dy = \int_0^d v_a \frac{R_0 - d}{R_0 - y}\, dy = v_a (R_0 - d) \log \frac{R_0}{R_0 - d} \quad \ldots\ldots(20)$$

Furthermore, designating the average velocity over the cross-section by v_m so that $q = v_m d$,

$$v_m d = v_a (R_0 - d) \log \frac{R_0}{R_0 - d} \quad \ldots\ldots\ldots\ldots\ldots\ldots(21)$$

or,

$$v_a = \beta\, v_m = \beta \frac{q}{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(22)$$

in which, the coefficient, β, is equal to,

$$\beta = \frac{1}{1 - \frac{d}{R_0}} \times \frac{1}{\log \frac{1 - d}{R_0}} \quad \ldots\ldots\ldots\ldots\ldots\ldots(23)$$

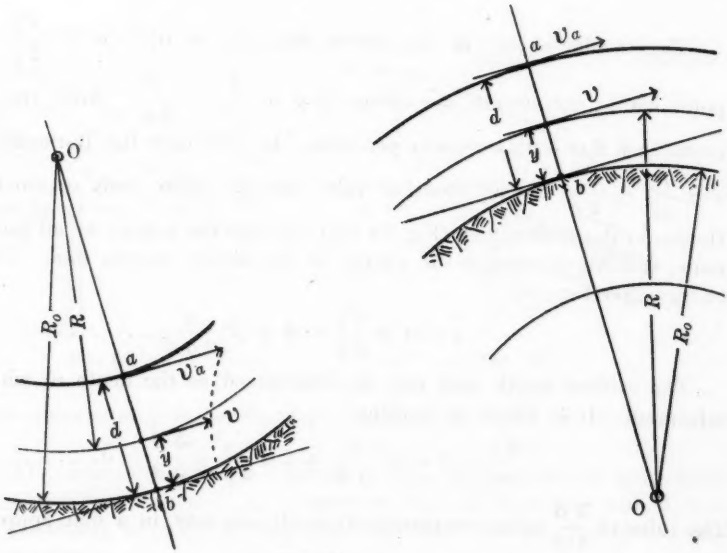

FIG. 24.

In the case of convex streaming (Fig. 24 (*b*)), analogous reasoning gives:

$$v = v_a \frac{R_0 + d}{R_0 + y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(24)$$

$$q = v_a (R_0 + d) \int_0^d \frac{d\, y}{R_0 + y} = v_a (R_0 + d) \log \frac{R_0 + d}{R_0} \quad \ldots\ldots(25)$$

and,

$$v_a = \beta\, v_m = \beta \frac{q}{d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(26)$$

in which, the value of the coefficient is,

$$\beta = \frac{\frac{d}{R_0}}{1 + \frac{d}{R_0}} \times \frac{1}{\log\left(1 + \frac{d}{R_0}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Numerical values of β are given in Table 8.

TABLE 8.—NUMERICAL VALUES OF β

$\frac{d}{R_0}$	0.10	0.20	0.30	0.40	0.50	0.60	0.75
β for concave streaming.....	1.063	1.12	1.20	1.303	1.44	1.64	2.16
β for convex streaming.....	0.953	0.925	0.88	0.848	0.82	0.795	0.77

The specific energy of the surface filament (at a) is $d + \frac{v_a^2}{2g}$. For any point within the stream, the energy is $y + \frac{P}{\Delta} + \frac{v^2}{2g}$. Now, free vortex is irrotational flow with a velocity potential. In such case the Bernoulli constant, $y + \frac{P}{\Delta} + \frac{v^2}{2g}$, has an identical value for the entire body of moving liquid. Hence, in the section, oab (Fig. 24 (b)), the specific energy at all points is the same, and thus is equal to the energy of the surface stream line. The specific energy, therefore, is:

$$\varepsilon = d + \frac{v_a^2}{2g} = d + \beta^2 \frac{q^2}{2g\,d^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

The critical depth may now be determined as the depth at which ϵ is a minimum. It is found by making

$$\frac{\partial \varepsilon}{\partial d} = 1 - \beta^2 \frac{q^2}{g\,d^3} + 2\beta \frac{q^2}{2g\,d^2} \frac{\partial \beta}{\partial d} = 0 \quad \ldots\ldots\ldots\ldots (29)$$

The value of $\frac{\partial \beta}{\partial d}$ being comparatively small, one may, in a first approximation, neglect the third member in Equation (29), with the result that,

$$d_{cr} = \sqrt[3]{\beta^2 \frac{q^2}{g}} = \beta^{\frac{2}{3}} \sqrt[3]{\frac{q^2}{g}} = \beta^{\frac{2}{3}} d_{cr} \quad \ldots\ldots\ldots\ldots (30)$$

The actual critical depth in concentric curvilinear motion is thus larger or smaller than the apparent critical depth, d'_{cr}, defined in Equation (17), by the value of the coefficient, $\beta^{\frac{2}{3}}$.

In curvilinear motion, as it occurs in actual practice, the various stream filaments usually have different centers of curvature. For example, at the extremity of the weir in Fig. 25, the surface point, c, has the greatest curvature, while the bottom filament is parallel to the floor at $R = r$. The average

energy of flow and the critical depth may still be represented by an expression similar to Equations (28) and (30), but the value of β will be different from that of Equation (23) or Equation (27), and will obviously lie between the value of β, corresponding to the extreme curvature of the surface filaments, and $\beta = 1$, corresponding to the bottom point. Finding the exact value of β presents in most cases a complicated and thankless venture. The foregoing analysis, however, clearly shows, that whenever the surface is curved, the real value of the critical depth is substantially different from the "apparent" depth corresponding to parallel movement and that such apparent value may not be used in hydraulic reasoning without due regard to the actual circumstances of flow. Combining Equations (28) and (30), the result with $d = d_{cr}$ is the value of $\varepsilon_{\text{min.}}$, the minimum possible contents of energy, which a discharge, q, carries in critical flow in curvilinear motion; in fact,

$$\varepsilon_{\text{min.}} = d_{cr}\left(1 + \beta^2 \frac{q^2}{2\,g\,d^3_{cr}}\right) = 1.5\,\beta^2_3 \sqrt[3]{\frac{q^2}{g}} \quad \ldots\ldots\ldots\ldots (31)$$

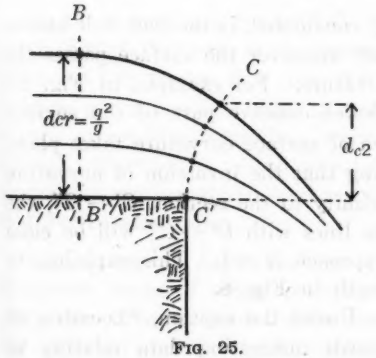

FIG. 25.

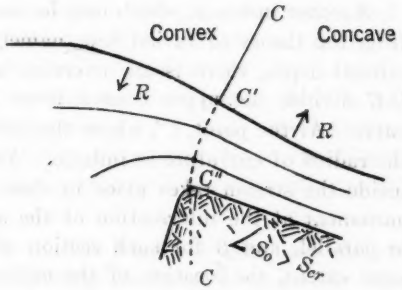

FIG. 26.

The minimum energy in parallel flow, $\varepsilon'_{\text{min.}} = 1.5 \sqrt[3]{\frac{q^2}{g}}$, is derived from Equation (31), assuming $\beta = 1$. Equation (31) shows that, in curvilinear motion, $\varepsilon_{\text{min.}}$ is different from $\varepsilon'_{\text{min.}}$ in parallel flow. It is greater with $\beta > 1$, when the streaming is concave, and less with $\beta < 1$ when the elementary filiments are convex.

With reference to the energy diagram, Fig. 19, this means, for example, that for convex flow the location of the point, C' (determined by d_{cr} and $\varepsilon_{\text{min.}}$), will be to the left and below Point C corresponding to parallel flow. Because of convex streaming, critical flow will occur with a smaller depth and with a smaller minimum contents of energy.

In this light, one may reconcile the apparent contradiction between the theory, according to which the minimum contents of energy were to be reached at the very end of the crest, and the observed fact, shown, for example, in Professor Woodburn's profiles, Series *A-B-C*, (Fig. 5), namely, that the critical depth, $d'_{cr} = \sqrt[3]{\frac{q^2}{g}}$, is found to lie at some distance from the

end of the crest. Referring to Fig. 25, it is seen that $d'_{cr} = \sqrt[3]{\frac{q^2}{g}}$ will be properly reached in Section B, where the parallel flow is supposed to end, but that flow may continue with a further loss of energy between Sections B and C, at the expense of a further reduction of energy, made possible by the fact that the surface becomes convex, with the curvature increasing as it approaches the end section, C.

Equations (13), (14) and (15) will properly apply to Section B, which usually is very close to the end of the crest.

It is further evident, in the light of the foregoing reasoning, that any device which may reduce the curvature, will result in bringing the locus of the apparent critical depth nearer the end of the crest. Thus, Professor Woodburn's profiles, Series $D\,A$ (Figs. 4 and 5), show how the addition of an apron, and particularly the backing up of tail-water, has the effect stated in the foregoing remarks.

Another instance, which may be usefully considered, is the fact well known from the theory of varied flow, namely, that whenever the surface passes the critical depth, there is an inversion of curvature. For example, in Fig. 26, C-C divides the upper convex from the lower concave part of the surface curve. At the point, C', where the inversion of surface curvature takes place, the radius of curvature is infinite. Assuming that the inversion of curvature inside the stream takes place in close proximity to the section, $C' - C''$, the movement at the intersection of the stream lines with $C' - C''$ will be close to parallel, and β for such section will approach $\beta = 1$. This explains, to some extent, the location of the critical depth in Fig. 8.

Waves on the Surface Over the Crest.—Under the caption, "Location of Critical Depths", Professor Woodburn records interesting data relating to the formation of waves on the surface over the weir. The theory of varied flow gives a sufficiently definite answer to this question. In general, it may be stated, that waves will form or will not be present, depending on whether the slope of the crest is smaller or larger than the so-called critical slope:

$$S_{cr} = \frac{g}{c^2} \times \frac{P}{b} \qquad (32)$$

Equation (32) determines S_{cr} as dependent on the depth, d, p being the wetted parameter, b, the top width, and c, the Chézy coefficient corresponding to a given depth of flow. For the rectangular flume with $b = 2$ ft., experimented upon, with C computed by means of the Ganguillet-Kutter formula with $n = 0.011$, the critical slope for the different depths is, as follows:

d, in feet.	S_{cr}.
0.1	0.0055
0.2	0.0044
0.4	0.0039
0.6	0.0039
0.8	0.0040
1.0	0.0043

The rule, as stated, follows from the study of the forms, which a surface assumes in the process of establishing a flow, given by Boussinesq.[6] With reference to the particular case under consideration, the deductions to be drawn from Boussinesq's most ingenious analysis, which are known to accord well with observation, would be as follows:

1.—When S is less than S_{cr} so that the moment which establishes itself is "tranquil", meaning a flow which, with the slope as given, would lead to a uniform movement with a depth greater than the critical, then the surface in the transition section will be undulated.

2.—When on the other hand, with S greater than S_{cr}, the movement which is in process of establishment is "rapid" (meaning that with the slope as given, the depth in uniform movement will be less than the critical), then the transition section will feature one inflected curve with no further undulations.

In Fig. 4, the slope in Series *D E*, is $S = 0.026$, far in excess of S_{cr}, as determined in the foregoing tabulation. Accordingly, there are no undulations.

On the other hand, Series *B G*, with a horizontal floor ($S = 0 < S_{cr}$), and partly Series *D D* and *D G*, register undulations. By coincidence the slope, 0.004, used in Series *D D* and *D G* is practically identical with the critical slope as given in the foregoing list. Therefore, the movement, which is in process of establishment, is a uniform movement with a depth equal to the critical. That is the reason why in these series the surface curve pendulates around the critical depth, and seems to appear repeatedly in the same profile in different locations.

D. D. Curtis,[7] Assoc. M. Am. Soc. C. E. (by letter).[7a]—The importance of various forms of weirs as means of measuring discharge is manifest. In particular, broad-crested weirs—because of several advantages, and perhaps more because of the comparative scarcity of data available—furnish especially interesting topics for investigation. This paper, therefore, is timely.

The writer was engaged in an experimental study of a round-crested weir, operated chiefly under condition of submergence, in the Hydraulic Laboratory of the University of Iowa during the summer of 1930. Although the work was undertaken for purposes different from those of Professors Woodburn and Webb, a comparison may prove of interest.

A weir, 18 in. high with a circular crest of 9-in. radius, was installed in a glass channel 2½ ft. wide. The up-stream face had a slope of 40°, and the down-stream face was a movable apron, set at angles varying from 10 to 45 degrees. The quantity of water was determined from readings of the head on a sharp-crested rectangular weir which had been previously calibrated volumetrically by Mr. M. E. Nelson, of the U. S. Army Engineer Office, at St. Paul, Minn. Velocity readings were taken with a Pitot tube designed and made in the laboratory. Water surface elevations were read with a movable point-gauge.

[6] "Essai sur la théorie des eaux courantes," Paris, 1877; also, "Théorie de l'écoulement tourbillonant," etc., 1897.

[7] Prof. of Mechanics and Hydraulics, Clemson Agri. Coll., Clemson College, S. C.

[7a] Received by the Secretary, January 12, 1931.

Fig. 27 shows the water surface profiles for a typical series of runs. In this set, two runs show depths greater than the critical and three were adjusted so that flow occurred at the critical depth in each. Sections at which critical depth existed are indicated by small circles.

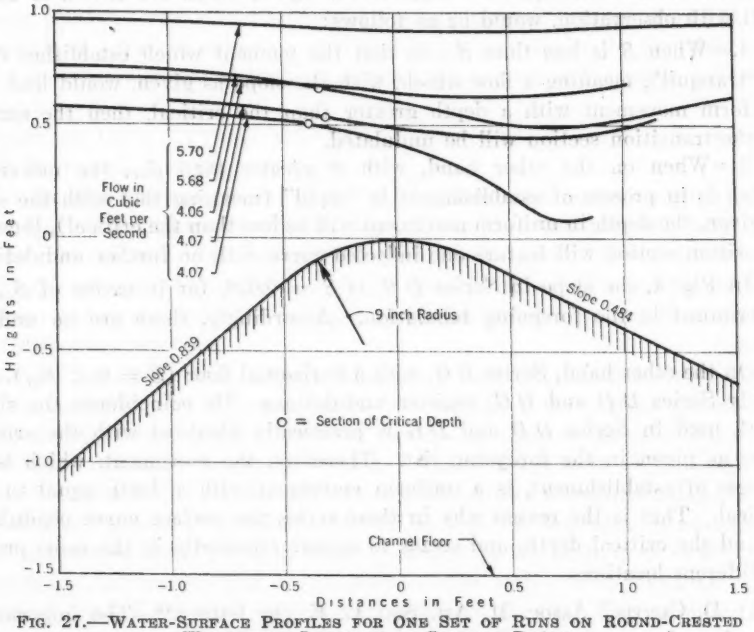

FIG. 27.—WATER-SURFACE PROFILES FOR ONE SET OF RUNS ON ROUND-CRESTED WEIR, WITH LOCATION OF CRITICAL DEPTH.

With the steepness of the aprons used in these experiments, it is indicated that the error in discharge, due to assuming critical depth to exist at the mean of the actual positions, might range up to ± 6 per cent. This, of course, is too inaccurate to be of value except where the most approximate results would be sufficient, and it demonstrates that this type of weir is not suitable for the determination of discharge from measurement of depth in one vertical plane if this depth is assumed to be the critical one.

Manipulation of tail-water height and quantity led the writer to believe that a condition of flow might persist through a greater degree of change on rising stage than on falling stage. For example, the unstable conditions accompanying critical flow might not become apparent so quickly when the water stage was rising as when it was falling, apparently indicating greater stability under rising than under receding waters. If this is true it is necessary for the experimenter to exercise some caution in adjusting the flow to meet any definite condition he desires, particularly if what he is attempting is a stage bordering on the critical.

It is to be hoped that these studies will be followed by others on this type of weir.

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DISCUSSIONS

STRESSES IN GRAVITY DAMS BY PRINCIPLE OF LEAST WORK

Discussion

BY MESSRS. FRED. W. ELY, AND M. H. GERRY, JR.

FRED. W. ELY,[12] M. AM. SOC. C. E. (by letter).[12a]—The author is to be commended for re-opening the discussion of stresses in gravity dams. By attacking the problem from a new angle he apparently has proved that the vertical pressures on a horizontal plane of a gravity dam do not vary in a linear relation. The popularity of the arch dam has withheld from the gravity dam the consideration that it merits, particularly as more than twenty years have passed since in 1909 the first English model tests suggested the incorrectness of the assumption of linear stress distribution. Attention also may be called to some recent tests of 8 by 8 by 12-in. prisms loaded at the edge of the middle third of the cross-section, made by Slater and Lyse[13] in connection with studies for determining the compressive strength of concrete in flexure. These are of interest in that tension was indicated on the face farthest from the loading point, particularly at the higher stresses, again indicating a non-linear stress distribution.

Further study may indicate that the author's stress equations require some modification. The results disclosed by the computations and graphs present some very startling results which suggest that Equation (15) may not be of the proper form to represent the stress relation correctly. At least, the computation of the total elastic work in some selected examples, using a basic equation of a different form, would throw additional light on the problem. As the author states, his proposed solution is only an approximation, limited by the equation assumed for n_y. This is not a general equation, even for the limited number of terms, since all the exponents of x are positive integers. There is some indication that fractional exponents might give results more in line with the results of theory and experiment in the flexure of beams.

NOTE.—The paper by B. F. Jakobsen, M. Am. Soc. C. E., was published in September, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1930, by the late William Cain, M. Am. Soc. C. E.; November, 1930, by Messrs. William Gore, L. J. Mensch, Johannes Skytte, and Lars R. Jorgensen; and January, 1931, by Messrs. C. A. P. Turner, Donald P. Barnes, and Anthony Hoadley.

[12] Chf. Designing Engr., Hydr. Dept., Aluminum Co. of America, Pittsburgh, Pa.

[12a] Received by the Secretary, November 29, 1930.

[13] "Compressive Strength of Concrete in Flexure as Determined from Tests of Reinforced Beams," by Willis A. Slater, M. Am. Soc. C. E., and Inge Lyse, Assoc. M. Am. Soc. C. E., *Journal*, Am. Concrete Inst., June, 1930, p. 831.

Referring to Fig. 6 the shape of the curve for bending stress with $w = 0$ appears to be somewhat inconsistent with the assumption of similar elastic properties in tension and compression. This assumption would logically presuppose a point of inflection at the place where the stress passes through zero, the two ends of the curve having similar curvature with respect to the X-axis. This would be in line with the stresses in a beam after the material has reached the elastic limit[14] and also with the **S**-shaped foundation deformation mentioned by Vogt.[15] A tendency in this direction is also indicated in the plotting of the results of the English tests, as shown on Section 5-5 of Fig. 8. The apparent inconsistency of the shape of this curve as well as that disclosed between Equations (27) and (36) might easily be due to the fact that Equation (15) cannot represent the stress relation properly.

The writer has worked through several numerical examples by the author's method, which indicate that, for a triangular section of an average specific weight, a base width of approximately 81% of the height would be required to prevent tension at the heel, even disregarding uplift. If this stress distribution is correct, practically no existing gravity dam is free from tensile stresses and, in the majority of gravity dams more than 150 ft. high, the tendency toward tension would be so great that the horizontal joints must have opened at some elevation, thereby assuring uplift which would tend to extend the cracks. The freedom from excessive leakage through the drainage wells which have been placed in many high dams, casts further doubt on the correctness of this stress distribution.

In order to obtain a more comprehensive insight into the stress distribution under the author's assumptions when applied to less usual types of section, the writer extended the method to include dams with a sloping up-stream face and has included in the general equations the effect of uplift. The additional nomenclature required is as follows (see Fig. 21):

e = minimum value of x for any given value of y (negative for the section shown in Fig. 21).

e_1 = batter of up-stream face $= \dfrac{e}{y}$.

k = batter of down-stream face $\left(\text{not equal to } \dfrac{b}{y} \text{ unless } e = 0\right)$.

ρ = unit water pressure in masonry units $= \dfrac{g}{w}$.

u = distance from up-stream face to point where uplift pressure is assumed equal to zero.

λ = proportional part of the base, perpendicular to the plane of the paper, over which uplift pressure is assumed to act.

The uplift condition assumed is that of full hydrostatic pressure at the heel, decreasing by a linear relation to zero at some distance from the heel where the drainage system is assumed to offer full relief. For the usual

[14] See Föppl, "Drang und Zwang," Second Edition, Vol. 2, p. 297; also, Timoshenko and Lessels, "Applied Elasticity," p. 54.

[15] "Ueber die Berechnung der Fundamentdeformation," Det Norske Videnkaps Akademi, Oslo, 1925; see, also, Love, "A Treatise on the Mathematical Theory of Elasticity," Fourth Edition, p. 140.

assumption of "full uplift", $u = b$ and $\lambda = 1$. It may be noted that e and e_1 would be positive in numerical value for an overhanging up-stream face.

An outline of the method of deriving the general equations, together with numerical results for one example, is given.

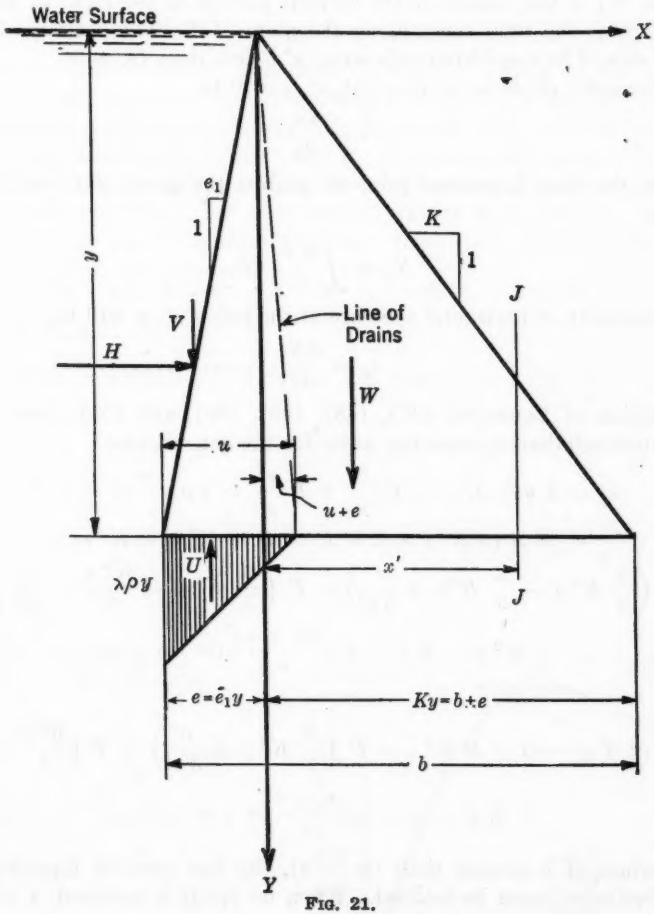

FIG. 21.

Based on the assumption of Equation (15), the vertical reaction of the foundation (or part of dam below any horizontal plane) at any point, x, y, is

$$p = A\,y + B\,x + C\,\frac{x^2}{y} + F\,\frac{x^3}{y^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (47)$$

To obtain the vertical stress in the concrete the uplift pressure at the point, $\left(= -\lambda\,\rho\,y\,\frac{u+e-x}{u}\right)$, must be added to the value of p in Equation (47). If $\lambda = 0$, $p = n_y$.

Considering the forces acting to the right of a vertical plane, *J-J*, the requirements for equilibrium of the severed portion are:

$$\int_{x'}^{K\,y} p\,dx + W_1 - \left[\int_{x'}^{u+e} \lambda\,\rho\,y\,\frac{u+e-x}{u}\,dx\right] = S_1 \ldots\ldots (48)$$

in which, W_1 is the weight of the severed portion to the right of the plane, *J-J*, and S_1 is the total shear along the plane, *J-J*. The portion within the brackets should be considered only when x' is less than $(u + e)$.

The intensity of shear at the point, x', y, will be,

$$t = \frac{dS_1}{dy} \ldots\ldots (49)$$

Again, the total horizontal pressure against the plane, *J-J*, will be represented by,

$$N_1 = \int_{x'}^{K\,y} t\,dx \ldots\ldots (50)$$

and the intensity of horizontal pressure at the point, x', y, will be,

$$n_x = \frac{dN_1}{dy} \ldots\ldots (51)$$

A solution of Equations (47), (48), (49), (50), and (51), gives the following stress relations in masonry units for the general case:

$$n_y = A\,y + B\,x + C\,\frac{x^2}{y} + F\,\frac{x^3}{y^2} - \lambda\,\rho\,y\,\frac{u+e-x}{u} \ldots\ldots (52)$$

$$n_x = A\,(3\,K^2\,y - 2\,K\,x) + B\,(2\,K^3\,y - K^2\,x) + C\left(\frac{3}{2}\,K^4\,y - \frac{2}{3}\,K^3 x + \frac{x^4}{6\,y^3}\right) + F\left(\frac{6}{5}\,K^5\,y - \frac{K^4\,x}{2} + \frac{3}{10}\,\frac{x^5}{y^4}\right) + K^2\,y - K\,x - \lambda\,\rho\,\frac{(u+e)^2}{u\,y}\,(u+e-x) \ldots\ldots (53)$$

and,

$$t = A\,(2\,K\,y - x) + B\,K^2\,y + C\left(\frac{2}{3}\,K^3\,y + \frac{x^3}{3\,y^2}\right) + F\left(\frac{K^4\,y}{2} + \frac{x^4}{2\,y^3}\right) + K\,y - x - \lambda\,\rho\,\frac{u+e}{u}\,(u+e-x) \ldots\ldots (54)$$

For values of x greater than $(u + e)$, the last term of Equations (52) to (54), inclusive, must be omitted. When no uplift is assumed, $\lambda = 0$, and the last term of each equation drops out.

The equations corresponding to Equations (3) and (4) are:

$$\int_{e_1\,y}^{K\,y} p\,dx = \frac{y^2}{2}\,[e_1\,(\rho + 1) - K + \lambda\,\rho\,u_1] \ldots\ldots (55)$$

and,

$$\int_{e_1\,y}^{K\,y} p\,x\,dx = -\frac{y^3}{6}\,[\rho + K^2 - {e_1}^2 - 2\,\rho\,{e_1}^2 - \lambda\,\rho\,u_1\,(u_1 + 3\,e_1)] \ldots (56)$$

in which, $u_1 = \dfrac{u}{y}$. The boundary conditions are:

At the toe,

$$t^2 = n_y\, n_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (57)$$

At the heel,

$$n_y\, e_1 + \rho\, y\, e_1\,(1 - \lambda) - t = 0 \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

and,

$$n_x + \rho\, y - t\, e_1 = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (59)$$

Equations (19) and (20) are modified only by changing the limits of integration with respect to x. These become $e_1\ y$ to $k\,y$ instead of 0 to b. These modified equations together with Equations (55) and (56) suffice to determine the parameters A, B, C, and F. A considerably greater amount of numerical computation is required in the solution than for the simple case of a vertical up-stream face, but it is lengthy rather than difficult.

The following equations show the stress relations in masonry units for a specific case, in which, $\lambda = 0$; $\dfrac{b}{y} = 0.80$; $k = 0.60$; $e_1 = -0.20$; $\rho = 0.416025$; and $m = 6$; thus,

$$n_y = -0.47905\ y - 1.7814\ x + 4.0734\,\frac{x^2}{y} - 2.4214\,\frac{x^3}{y^2} \ldots\ldots\ldots (60)$$

$$n_x = -0.36102\ y + 0.18650\ x + 0.67890\,\frac{x^4}{y^3} - 0.72643\,\frac{x^5}{y^4} \ldots\ldots (61)$$

and,

$$t = -0.18650\ y - 0.52095\ x + 1.3578\,\frac{x^3}{y^2} - 1.2107\,\frac{x^4}{y^3} \ldots\ldots\ldots (62)$$

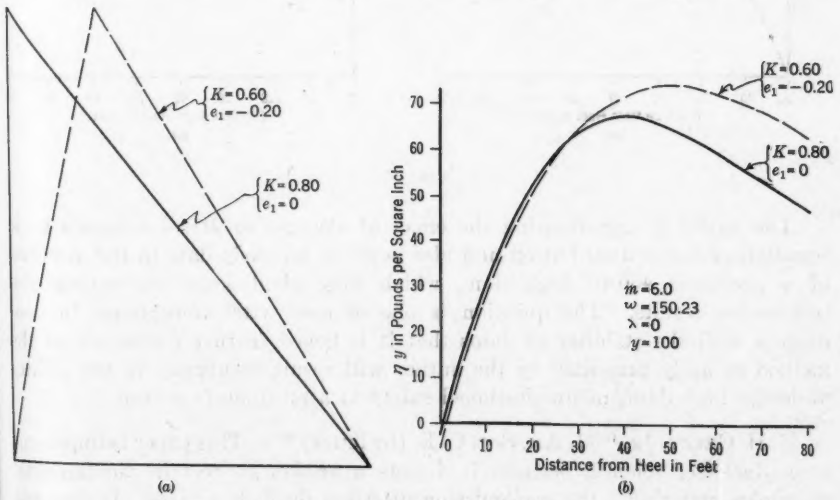

FIG. 22.

The stresses from Equation (60) for $y = 100$, reduced to pounds per square inch, are plotted to scale in Fig. 22, which also shows the n_y-curve for a section of identical base width, but with a vertical up-stream face. A com-

parison of the two sections is shown in the upper part of the diagram. Fig. 23 shows a plotting of the horizontal stress and shearing stress for the two sections.

It is interesting to note the close correspondence of the two vertical stress curves in Fig. 22(*b*), throughout the up-stream 30 ft. of base width. The conditions at the heel are more favorable for the inclined up-stream face as compared to a vertical face than would be the case with linear stress distribution.

If the stress distribution assumed in Equation (15) is confirmed by further theoretical analysis and experiment, gravity dams with an appreciable up-stream slope would appear worthy of more consideration because of their greater resistance to sliding on the foundation.

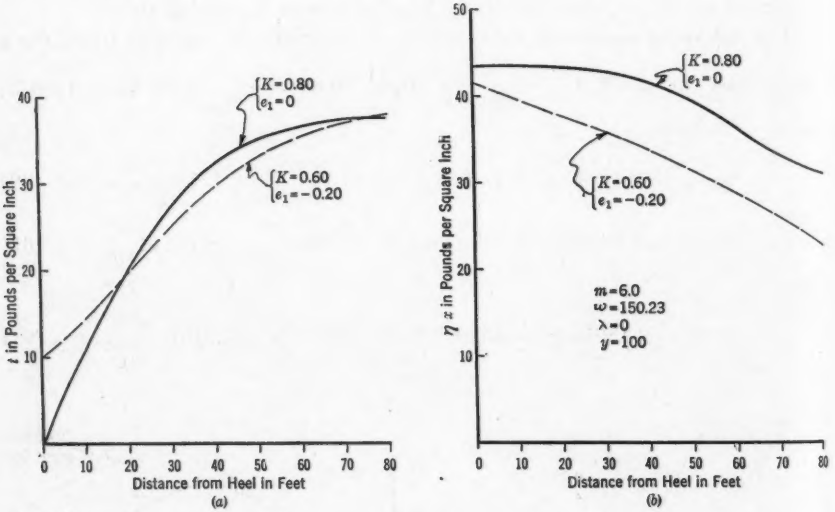

FIG. 23.

The writer is investigating the effect of changes in Mr. Jakobsen's basic equation of stress distribution and also hopes at an early date to test a model of a projected 450-ft. high dam, which may yield some interesting and instructive results. The question is one of such vital importance in connection with the stability of dams that it is hoped further discussion of the method so aptly presented by the author will result eventually in the ability to design high dams of unquestioned safety at a minimum expense.

M. H. GERRY, JR.,[16] M. AM. SOC. C. E. (by letter).[16a]—This paper is important to engineering science, because it directs attention to certain fundamental principles underlying the transmission of stress through a mass. It does not of itself, however, offer a solution of stress distribution in actual dams, but is rather a mathematical discussion based on an assumed state of unknown facts.

[16] Cons. Engr., San Francisco, Calif.

[16a] Received by the Secretary, December 9, 1930.

Nevertheless, it is of value because it points the way to further progress and calls to notice some common errors.

The manner in which stress acts on and through bodies of various kinds is unknown in any exact sense and no general solution is possible until far more is learned of the molecular state of matter. Moreover, stress acts in three dimensions and not only in two, as assumed for convenience of computation in this paper. The mathematical difficulties of the problem are very great, but by no means insuperable. The crux of the situation is the insufficiency of knowledge concerning physical facts and this can be overcome only by long and extensive laboratory research; there is no short cut.

The theory of "least work" is apparently a sound and scientific method of approach, but more than mere assumptions must be available before general conclusions of value can be determined from its application.

The present methods of treating stress distribution in structures are well known to be empirical and approximate only, and it is most important that this fact be fully and constantly recognized by designing engineers.

The so-called elastic theory is such a case and it should never be applied outside the known limits of actual experiment. A definite "modulus of elasticity" is a mathematical fiction in so far as it represents a positive condition of a material. Over any considerable range it is quite commonly a variable and at best the modulus is merely a constant of experiment, its correctness being limited to the special physical conditions and dimensions at which it was determined.

Concrete is elastic in only a very limited way. It is not a distinct material, but rather an artificial conglomerate of which little is known in a scientific sense. It has no true elastic limit, and it flows and yields inelastically under moderate loadings. At all ages, concrete when passing from a dry to a wet state expands and its strength and elasticity materially change, while time, temperature, stress, and vibration further alter its character. Thus, it is idle to attempt an exact general analysis of stress distribution in an uncertain mass, such as a dam, as long as the profession remains intensely ignorant of the basic facts concerning stress in highly uniform and far more elastic materials.

At present, the design of gravity dams is purely empirical; attempts to cloak certain phases in a mathematical fog have only resulted in serious errors, causing many failures. Unfortunately, there has been a tendency to follow a fetish and to place total reliance on a margin of safety in one particular only, while overlooking entirely, weakness in other directions. All modern gravity dams of concrete, having triangular sections and bases of more than two-thirds the height, provide good margins of safety as to stress, but all too frequently they are close to the danger line in other respects. The proper limiting of stress in gravity dams is important, and, in future very high dams, it may control; but for present-day structures, it does not enter to the same degree as other important conditions for safety.

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DISCUSSIONS

THEORY OF SIMILARITY AND MODELS

Discussion

BY MESSRS. A. V. KARPOV, AND JASPER O. DRAFFIN

A. V. KARPOV,[13] M. AM. SOC. C. E. (by letter).[13a]—The use of models in connection with engineering studies is not a recent development; it has become extensive only in the past few years. The advent of aeronautics and the successful use of models in the investigations of the complicated problems of this new branch of engineering have brought out the importance of model studies. This is true not only in aeronautics but in many other fields of engineering endeavor and started the general trend of the extensive use of models in involved engineering studies and investigations which is evident to-day.

The excellent presentation of the underlying theoretical principles of model studies by the author will probably do much toward the removal of the misconceptions that are so common in the model work of the past. Mr. Groat discusses the models involving dynamic problems, which are naturally the more complicated.

In connection with the arch dam investigation that is being conducted by the Company with which the writer is connected, it was necessary to determine the conditions that will govern the building of a model of an arch dam, in order to study the structural and design problems arising in this investigation. This work stresses not only the theoretical aspects of the problem, but the practical ones as well, and is included in this discussion in its entirety.

Similarity Conditions Between an Arch Dam Model and Its Prototype.—Past experience in solving complicated engineering problems by model studies not only justifies the use of models, but shows that in many cases the only way in which reliable results can be obtained is by making proper model studies.

The first step in making such study is to determine what, in the engineering sense, is a model. A very common conception, even in engineering circles, is that any representation of the prototype in an arbitrary scale is already a model; but from the engineering point of view, a scalar representation of the prototype can be called a model only when, from a study of its behavior, conclusions can be derived that will disclose the behavior of the prototype. This

NOTE.—The paper by Benjamin F. Groat, M. Am. Soc. C. E., was published in October, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: January, 1931, by Messrs. H. Bateman and W. P. Roop.

[13] Designing Engr., Hydr. Dept., Aluminum Co. of America, Pittsburgh, Pa.

[13a] Received by the Secretary, November 29, 1930.

means that a certain relationship is to exist between the behavior of the model and prototype, which necessarily must be of a rather simple nature and which is brought about by the fulfillment of the similarity conditions.

These similarity conditions are governed by the purpose for which the model study is to be made and may cover a wide range of different requirements. In a number of simpler problems the only similarity condition to be met is the proper scalar representation of the prototype; in more complicated problems additional requirements will be included and there are still other problems in which the similarity conditions will require that the model is not a true scalar representation of the prototype. In many cases the relationship between the properties of the material or materials that are used in the prototype and in the model is covered by similarity requirements that may be of no less importance than requirements covering the geometrical relationship between the shapes of the prototype and the model.

The scope of the similarity conditions depends not only on the problem itself, but often, to a larger extent, on the kind of information that is to be obtained from the model study. The similarity conditions may be simple if the study is restricted to the influence of some main factors. With the increase of the number of factors that are to be included in a study the similarity conditions increase very rapidly in number and in scope.

In studies of the more complicated structural problems practically every model represents a certain compromise between the rigid theoretical requirements and some practical considerations.

The cost of the model study measured in terms of time and money is the most important of all the practical considerations; if it could be neglected, the best possible model on which to make the studies would be the prototype itself.

The following analysis of the similarity conditions of a model of a simply supported concrete beam will serve as an illustration:

A.—If the purpose of the study is to determine the deflections and stresses of a prototype beam under uniform or concentrated loads of such character and magnitude that the deflections due to shear and the dead weight of the beam can be neglected, then the similarity conditions can be formulated very simply:

(*a*) The model is to be a true scalar representation of the prototype.

(*b*) The material of the model is to be such that, in the range of stresses of the model, the variations of the modulus of elasticity are similar to the variations of the modulus of elasticity of the material of the prototype in its corresponding range of stresses.

(*c*) The loadings of the model are to be proportional to the loadings of the prototype, but no limitations apply to the value of the model loadings.

B.—If the same study is to be made under such conditions that the dead weight of the beam cannot be neglected, then the similarity requirements introduce the following additional conditions:

(*d*) The specific gravity of the material of the prototype and of the beam should be equal.

(*e*) The loadings of the model are to be not only proportional to the loadings of the prototype, but the ratio of the loadings of the prototype and model is to be equal to the square of the linear scale of the model.

C.—If the study is to be extended so that it includes the deflections due to shear, then the additional similarity condition will be:

(*f*) The ratio of the modulus of elasticity in shear and of the modulus of elasticity in flexure of the model material within the range of the stresses of the model is to be the same as that of the material of the prototype within its range of stresses. In other words, the Poisson's ratios of the model and prototype materials are to be equal at corresponding stresses.

D.—If the same study is to be made on a reinforced concrete beam, then the representation of the reinforcement in the model can be so inconvenient that the practical compromise would be to make a model that is a scalar representation of the prototype in its length only. Such a model would have cross-sections which would not be scalar representations of the prototype. It would represent the reinforcement by a proper increase of the size of the part of the beam section that includes the steel.

E.—The practical compromise that probably will be necessary to make Model Studies A to D possible would be to simplify the similarity conditions by assuming that the modulus of elasticity and Poisson's ratio of the material of the prototype are constant and to choose a material for the model that is fairly uniform, the modulus of elasticity and Poisson's ratio of which can be assumed as constant.

F.—No restrictions apply to the linear scale of the model and to the value of the modulus of elasticity of model material, but for models that satisfy the similarity conditions in Model Study B, the following conclusions can be derived:

(*g*) The deflections of the model are proportional to the square of the linear scale multiplied by the ratio of the moduli of elasticity of the prototype and model materials.

(*h*) The stresses of the model are proportional to its linear scale.

G.—If it is desirable that the geometrical resemblance of the model and prototype is strictly retained after the application of the load, an additional condition will be imposed:

(*i*) The ratio of the moduli of elasticity of the materials of the prototype and model is to be equal to the linear scale of the model.

H.—If the model is to be tested to destruction then, again, an additional condition will be:

(*j*) The ratio of the moduli of rupture of the materials of the prototype and model is to be equal to the linear scale of the model.

An arch dam represents a more complicated problem than a simply supported beam and, keeping in mind the purpose for which the model study is to be made, the problem can be stated in a general form, as follows:

Determine the similarity conditions of an arch dam model in which the deflections and stresses are to have some simple relationship with the deflections and stresses of the prototype under conditions that are well below the destruction point of the prototype.

A certain amount of judgment must be exercised to classify and give the proper weight to the different similarity requirements and the most practical

way would be to formulate the theoretical similarity conditions and, later, to bring them into agreement with the practical requirements. No single method is known that would determine all the similarity conditions and the entire problem is to be considered from different points of view. The first and most formal will be the conclusions that can be derived from the application of the dimensional theory, but this theory alone by no means covers all similarity conditions; some additional conditions that are not based on this theory, must be derived.

Similarity Conditions Derived by Application of Dimensional Theory.— The dimensional theory can be applied to the studies of an arch dam model, as follows: The deflection can be expressed as,

$$\delta_p = \psi' (\rho_p, \gamma_p, E_p, S_p, g, l_p) \ldots\ldots (63)$$

and the stresses,

$$f_p = \psi (\rho_p, \gamma_p, E_p, S_p, g, l_p) \ldots\ldots (64)$$

in which,

ψ' and ψ = some unknown functions;
ρ_p = mass per unit of volume of the material of the dam;
γ_p = mass per unit of volume of the loading liquid of the dam (water);
E_p = modulus of elasticity in flexure of the material of the dam;
g = acceleration due to gravity;
l_p = length scale; and
S_p = modulus of elasticity in shear of the material of the dam.

If μ_p is Poisson's ratio of the material of the dam, then:

$$S_p = \frac{E_p}{2 (1 + \mu_p)} \ldots\ldots (65)$$

and since μ_p is not a dimensional quantity, the dimensional Equations (63) and (64) can be written:

$$\delta_p = \psi' (\rho_p, \gamma_p, E_p, g, l_p) \ldots\ldots (66)$$

and,

$$f_p = \psi (\rho_p, \gamma_p, E_p, g, l_p) \ldots\ldots (67)$$

Expressing all the quantities of Equations (66) and (67) in terms of mass, M, length, L, and time, T, then:

$$\rho_p = \gamma_p = M L^{-3} \ldots\ldots (68)$$

$$E_p = M L^{-1} T^{-2} \ldots\ldots (69)$$

$$g = L T^{-2} \ldots\ldots (70)$$

$$l_p = L \ldots\ldots (71)$$

The deflections have the dimensions, L, and from Equation (66) the dimensional equation will be,

$$\delta_p = \rho_p^{\,x} \gamma_p^{\,y} E_p^{\,z} g^u l_p^{\,v} \ldots\ldots (72)$$

making the necessary substitutions:

$$L = L^{(-3x-3y-z+u+v)} M^{(x+y+z)} T^{(-2z-2u)} \ldots\ldots (73)$$

The following equations can be written by equalizing the powers of L, M, and T:

$$1 = -3x - 3y - z + u + v \quad \ldots\ldots(74)$$

$$0 = x + y + z \quad \ldots\ldots(75)$$

and,

$$0 = -2z - 2u \quad \ldots\ldots(76)$$

Solving these equations:

$$z = -u \quad \ldots\ldots(77)$$

$$v = 1 + u \quad \ldots\ldots(78)$$

and,

$$x + y = u \quad \ldots\ldots(79)$$

Substituting in Equation (72),

$$\delta_p = l_p \left(\frac{\gamma_p\, g\, l_p}{E_p}\right)^u \left(\frac{\rho_p}{\gamma_p}\right)^x \quad \ldots\ldots(80)$$

Assuming that the material follows Hooke's law and that no buckling or other disturbances which would distort the deflections occur, then δ_p will be proportional to $\frac{1}{E}$ and, consequently, from Equation (80), $u = 1$, and,

$$\delta_p = \frac{\gamma_p\, g}{E_p}\, l_p^{\,2}\, \phi'_p \left(\frac{\rho_p}{\gamma_p}\right) \quad \ldots\ldots(81)$$

in which, ϕ'_p is an unknown function.

Using the subscript, m, the deflections of the model can be written similarly to Equation (81),

$$\delta_m = \frac{\gamma_m\, g}{E_m}\, l_m^{\,2}\, \phi'_m \left(\frac{\rho_m}{\gamma_m}\right) \quad \ldots\ldots(82)$$

From Equations (81) and (82),

$$\frac{\delta_m}{\delta_p} = \left(\frac{l_m}{l_p}\right)^2 \frac{\gamma_m}{\gamma_p}\, \frac{E_p}{E_m}\, \frac{\phi'_m \left(\frac{\rho_m}{\gamma_m}\right)}{\phi'_p \left(\frac{\rho_p}{\gamma_p}\right)} \quad \ldots\ldots(83)$$

By making the ratio, $\frac{\rho_m}{\gamma_m}$, in the model the same as the corresponding ratio in the dam (that is, the ratio of the specific gravities of the material of the structure to the loading fluid), it will become a model constant, and,

$$\frac{\delta_m}{\delta_p} = \left(\frac{l_m}{l_p}\right)^2 \frac{\gamma_m}{\gamma_p}\, \frac{E_p}{E_m} \quad \ldots\ldots(84)$$

because,

$$\frac{\rho_m}{\gamma_m} = \frac{\rho_p}{\gamma_p} \quad \ldots\ldots(85)$$

In the same way from Equation (67),

$$f_p = \rho_p^{\,x}\, \gamma_p^{\,y}\, E_p^{\,z}\, g^u\, l_p^{\,v} \quad \ldots\ldots(86)$$

and since the dimension of f_p is $M\,L^{-1}\,T^{-2}$,

$$f_p = E_p \left(\frac{\gamma_p\, g\, l_p}{E_p}\right)^u \phi_{fp}\left(\frac{\rho_p}{\gamma_p}\right) \quad \ldots\ldots (87)$$

in which, ϕ_{fp} is an unknown function.

The stresses do not depend on E_p. Consequently, $u = 1$, and,

$$f_p = \gamma_p\, g\, l_p\, \phi_{fp}\left(\frac{\rho_p}{\gamma_p}\right) \quad \ldots\ldots (88)$$

For the model, similarly,

$$f_m = \gamma_m\, g\, l_m\, \phi_{fm}\left(\frac{\rho_m}{\gamma_m}\right) \quad \ldots\ldots (89)$$

and,

$$\frac{f_m}{f_p} = \frac{\gamma_m}{\gamma_p}\,\frac{l_m}{l_p}\,\frac{\phi_{fm}\left(\frac{\rho_m}{\gamma_m}\right)}{\phi_{fp}\left(\frac{\rho_p}{\gamma_p}\right)} \quad \ldots\ldots (90)$$

Considering, again, $\dfrac{\phi_{fm}\left(\frac{\rho_m}{\gamma_m}\right)}{\phi_{fp}\left(\frac{\rho_p}{\gamma_p}\right)}$ as a model constant,

$$\frac{f_m}{f_p} = \frac{l_m}{l_p}\,\frac{\gamma_m}{\gamma_p} \quad \ldots\ldots (91)$$

Equations (84) and (91) give the similarity conditions of the model that can be derived from the dimensional theory.

Introducing the linear scale,

$$R = \frac{l_m}{l_p} \quad \ldots\ldots (92)$$

These equations can be written:

$$\delta_m = R^2\,\frac{\gamma_m}{\gamma_p}\,\frac{E_p}{E_m}\,\delta_p \quad \ldots\ldots (93)$$

and,

$$f_m = R\,\frac{\gamma_m}{\gamma_p}\,f_p \quad \ldots\ldots (94)$$

In order to arrive at these equations the two following assumptions were made, without which the similarity conditions would be so complicated that the model would be impractical:

(1) The materials of the prototype and model shall follow Hooke's law, which means that the modulus of elasticity, E, is constant in the prototype and in the model.

(2) The ratio of the specific gravities of the material of the prototype and its loading fluid, $\frac{\rho_p}{\gamma_p}$, is equal to the same ratio of the model, $\frac{\rho_m}{\gamma_m}$.

Similarity Conditions Covering Non-Dimensional Factors.—In applying the dimensional theory the factors that have no dimension cannot be covered,

and the similarity conditions for such factors have to be investigated separately.

Poisson's ratio is a non-dimensional factor and is to be investigated by a different method. This ratio influences the conditions of the dam in two ways: First, directly, by the introduction of stresses at right angles to the stress developed by the applied force; and, second, due to the dependence of the modulus of elasticity in shear on Poisson's ratio and the consequent dependence of the shear deflections on this ratio. These factors will be considered separately.

The direct influence of Poisson's ratio can be handled in the following way. Due to Poisson's ratio, μ_p, of the dam material, a stress, f_p, will produce in a restrained unit cube of the dam a stress, f'_p, at right angles to it,

$$f'_p = \mu_p f_p \quad \ldots\ldots (95)$$

and similarly for the model,

$$f'_m = \mu_m f_m \quad \ldots\ldots (96)$$

To retain the similarity conditions between the dam and model it is necessary that,

$$\frac{f'_p}{f'_m} = \frac{f_p}{f_m} \quad \ldots\ldots (97)$$

and, consequently,

$$\mu_p = \mu_m \quad \ldots\ldots (98)$$

The indirect influence of Poisson's ratio can be treated in a similar manner:

The deflection of any part of the prototype due to shear will be,

$$\delta_{sp} = \frac{1}{S_p} \int s_p \, dl_p \quad \ldots\ldots (99)$$

in which, s_p is unit shear; and dl_p, differential distance.

Similarly, for the model,

$$\delta_{sm} = \frac{1}{S_m} \int s_m \, dl_m \quad \ldots\ldots (100)$$

From Equations (92) and (94),

$$dl_m = R \, dl_p$$

$$s_m = R \frac{\gamma_m}{\gamma_p} s_p$$

Substituting these values in Equation (100),

$$\delta_{sm} = \frac{1}{S_m} (R)^2 \frac{\gamma_m}{\gamma_p} \int s_p \, dl_p$$

and,

$$\frac{\delta_{sm}}{\delta_{sp}} = \frac{S_p}{S_m} (R)^2 \frac{\gamma_m}{\gamma_p}$$

To retain the similarity between the model and prototype, $\frac{\delta_{sm}}{\delta_{sp}} = \frac{\delta_m}{\delta}$, and, from Equations (93) and (65),

$$\frac{E_p}{E_m} = \frac{S_p}{S_m} = \frac{2\,(1 + \mu_m)}{2\,(1 + \mu_p)} \frac{E_p}{E_m}$$

and, $\mu_p = \mu_m$. This will add the condition:

(3) Poisson's ratios of the prototype and model materials are to be equal at corresponding stresses.

Similarity Conditions Derived from Geometrical Similarity.—In order to have the prototype and model similar in their geometrical shape before and after the loadings are applied, the linear deflections of the model are to be in the same linear scale as the model itself. Therefore, $\delta_m = R\,\delta_p$. Introducing this relation in Equation (93),

$$R\,\frac{\gamma_m}{\gamma_p}\,\frac{E_p}{E_m} = 1 \qquad (101)$$

or,

$$\frac{E_m}{E_p} = R\,\frac{\gamma_m}{\gamma_p} \qquad (102)$$

which introduces the additional condition:

(4) The ratio of the moduli of elasticity of the model and prototype is to be equal to the scale of the model multiplied by the ratio of the specific gravities of the loading liquids of the model and prototype.

Similarity Conditions Covering Discontinuities.—An actual arch dam cannot be built as a continuous structure; depending on the methods by which it is built, it has a number of construction joints which may become planes of discontinuity.

The construction joints of the dam are to be represented in the model so as to maintain the similarity between the model and dam.

The dam has horizontal and vertical construction joints. The horizontal construction joints are kept uneven and rough across the block in order to insure a better bond between the successive layers of concrete and as long as the dam is not loaded no separation or opening of these joints can take place. The vertical construction joints divide the dam into separate blocks and although bonding boxes are provided, these joints are much more definite than the horizontal joints, and when the dam is not loaded they may open due to the contraction of the concrete. The horizontal construction joints can carry a limited tension that is much less than the tension that can be carried in a concrete block between the joints, but the vertical joints cannot carry any tension. In conformity with these conditions the horizontal and vertical joints of the model are to be treated differently.

The horizontal joints are construction joints only, and it can be assumed that after the dam is constructed no measurable horizontal joints are left. Even if the dam is loaded far below its destruction point, the tension stresses in some of the joints can be higher than the ultimate tension that can be carried over the joint. In this case, a rupture of such joints will take place starting at the surface and extending to some point at which the tension stress is low enough to be carried by the joint.

In indicating the ultimate unit tensile stresses that can be carried over the joints of the prototype and model by f_{jp} and f_{jm}, respectively, the requirement would be that,

$$\frac{f_{jm}}{f_{jp}} = \frac{f_m}{f_p} \qquad (103)$$

and from Equation (94),

$$\frac{f_{jm}}{f_{jp}} = R\,\frac{\gamma_m}{\gamma_p} \qquad (104)$$

which adds the additional condition:

(5) The ratio between the ultimate unit tensile stresses that can be carried by the horizontal joints of the model and prototype is to be equal to the linear scale of the model multiplied by the ratio of the specific gravities of the loading liquids of the model and prototype.

The vertical joints of a dam can have a physical dimension and, in such case, have to be represented in the model as joints of a certain size.

Considering the vertical joints of an actual dam, the action of these joints can be represented by two steps: First, Poisson's ratio of the dam material causes a part, b'_p, of the width of joint, b_p, to be closed; and, second, the remaining part of the joint, b''_p, will increase the deflection of the arch.

Considering a block of the length, s_p,

$$b'_p = \mu_p\,\frac{1}{E_p}\,f_p\,s_p \qquad (105)$$

and, similarly, for the model,

$$b'_m = \mu_m\,\frac{1}{E_m}\,f_m\,s_m \qquad (106)$$

and,

$$b'_m = \frac{\mu_m}{\mu_p}\,\frac{E_p}{E_m}\,\frac{f_m}{f_p}\,R\,b'_p \qquad (107)$$

Substituting Equation (94),

$$b'_m = R^2\,\frac{E_p}{E_m}\,\frac{\gamma_m}{\gamma_p}\,\frac{\mu_m}{\mu_p}\,b'_p \qquad (108)$$

If the action of the remaining part of the joint is to be represented in the model so as to result in deflections of the model that are of the same scale as the deflections due to elastic deformations, then, from Equation (93),

$$b''_m = R^2\,\frac{E_p}{E_m}\,\frac{\gamma_m}{\gamma_p}\,b''_p \qquad (109)$$

and, since $b'_m + b''_m = b_m$,

$$b_m = R^2\,\frac{E_p}{E_m}\,\frac{\gamma_m}{\gamma_p}\left[b''_p + \frac{\mu_m}{\mu_p}\,b'_p\right] \qquad (110)$$

Remembering Equation (98),

$$\frac{b_m}{b_p} = R^2\,\frac{E_p}{E_m}\,\frac{\gamma_m}{\gamma_p} \qquad (111)$$

which adds the additional condition:

(6) The ratio of the width of the vertical construction joints in the model and prototype is to be equal to the square of the scale of the model multiplied by the ratio of the specific gravities of the loading liquids and by the inverse ratio of the moduli of elasticity of the model and prototype.

Similarity Conditions Covering Foundations and Abutments.—The elastic deformations of an arch dam are by no means limited to the dam itself, but

are carried through the foundations and abutments. The previously developed formulas and statements concerning the dimensional theory, non-dimensional factors, and geometrical shape can also be applied to the foundation and abutments. It is obvious, however, that the size of the elastic foundations that are practically infinitely large as compared with the size of the dam cannot be represented in a practical model, and some compromise must be made in the very beginning if the model is to be feasible.

The theoretical investigation of foundation deflections made by Fredrik Vogt, Assoc. M. Am. Soc. C. E., entitled "Ueber die Berechnung der Fundamentdeformation", disclosed the fact that the action of the infinitely large elastic rock foundation can be closely represented by an imaginary block of definite dimensions, made of elastic material and rested on a fixed non-elastic and non-yielding base.

The dimensions of such a foundation block and its elastic properties depend on the elastic properties of the foundation rock. The investigations available in this respect are very meager as far as the elastic properties of the ground rock and the relationship between these properties and the sizes and elastic properties of the imaginary foundation block are concerned. It can be stated in general terms that the following condition should be added:

> (7) The action of the infinitely large foundation rock may be represented by a block of elastic material of some definite dimensions, which has a fairly low modulus of elasticity so that by supporting it on a base of a material of a high modulus of elasticity, the conditions of an elastic foundation block supported on an unyielding non-elastic base can be represented.

Similarity Conditions Covering Influence of Temperature Changes.—In the study of the stresses and deflections of the model due to different temperatures the following simple relationships are obvious:

$$c_p T_p = c_m T_m \quad \ldots\ldots (112)$$

and,

$$T_m = \frac{c_p}{c_m} T_p \quad \ldots\ldots (113)$$

in which, c_p and c_m are the coefficients of thermal expansion of the prototype and model; T_p is the difference in temperature of the material of the dam between the time the dam was built and the time under consideration; and T_m, the corresponding temperature difference of the model.

The only theoretical complication that would enter is, that consideration should be given not only to the coefficients of thermal expansion of the materials of the prototype and models, but also to the materials of the actual foundation and the imaginary foundation block; but that is a practical impossibility, and being of no importance, is to be neglected. The additional condition to be included can be formulated, as follows:

> (8) The ratio of the temperature differences of the model material, between the time of testing and building of the model, and of the temperature difference of the material of the prototype, between the time for which the investigations are made and the time of the building of

the prototype, should be equal to the ratio of the coefficients of thermal expansion of the materials of the model and the prototype, provided that these coefficients are sufficiently uniform to be assumed as constant.

Type of Model.—These similarity conditions are general, and any model of an arch dam that is to be used to study the stresses and deflections should satisfy them.

An attempt to build a model covered by this set of similarity conditions will meet with many obstacles, and in order to build one with a reasonable expenditure of time and money a number of additional compromises are necessary. These must be determined, analyzing separately each of the similarity conditions as applied to a particular problem to be investigated.

The dam described in this discussion is the Calderwood Arch Dam on the Little Tennessee River, which is about 200 ft. high and about 800 ft. long at the crest. Two different kinds of models were considered: First, one in which, at a number of points, the deflections of the down-stream surface and the strains at the down-stream and up-stream surfaces of the model could be measured directly; and, second, a model in which some information could be obtained about the distribution of stresses through the thickness of the model.

The first type would represent a model along the more conventional lines; the second could be obtained by the use of a modification of the photo-elastic method of stress determination. The second model would look very attractive if practical results could be obtained, having the following advantages:

(*a*) Determinations of stress through the thickness of the model as against the determination at the up-stream and down-stream surfaces of the model only.

(*b*) The possibility of the use of a smaller model and, consequently, a possible saving in cost.

The disadvantages of such a model can be summarized as follows:

(*c*) This method has not been used on three-dimensional models. The possibilities are that it would require considerable time to make the improvements or modifications that would be necessary in order to adapt the method to the conditions of an arch dam model.

(*d*) The usual transparent materials used in this method have a high modulus of elasticity and the proper reproduction in small scale of the details of the prototype would be difficult.

(*e*) Considerable time would be necessary to determine by test and trial the material that would be suitable for such a model.

Considering the uncertain features of the photo-elastic method, it was decided to build a model of the Calderwood Dam according to the first method, and the following discussion is based on this decision.

Loading Liquid.—One of the most important points, which governs all other conditions, to a considerable degree, is the choice of the loading liquid.

Condition (2) requires that the ratio of the specific gravities of the loading fluids of the model and prototype shall be equal to that of the materials of the model and prototype. This requirement is fundamental, because if it is neglected, the relationship between the arch and gravity action of the model will be distorted and the study of the behavior of the dam will be made difficult and uncertain due to the necessity of correcting the results to take care of a number of secondary influences that will be introduced. From the practical

point of view, the choice of the loading liquid of the model is rather limited. Either the same liquid must be used as in the prototype (which is water, the specific gravity of which is equal to 1), or the next reasonable possibility would be the use of mercury which has the specific gravity of 13.6.

Using water, the specific gravity of the material of the model should be about 2.4, the same as for concrete, the material of which the dam is constructed. Using mercury, the specific gravity of the material of the model will be $2.4 \times 13.6 = 32.84$. That would practically exclude the use of mercury, because no known material has such a high specific gravity, and the only way to approximate these conditions would be by the use of some artificial method of loading the model with additional loads or springs.

The choosing of water as the loading liquid will determine the specific gravity of the material of the model equal to about 2.4 and will simplify the similarity conditions considerably since $\frac{\gamma_m}{\gamma_p} = 1$, and Equations (93), (94), (102), (104), and (111) can be written:

$$\delta_m = R^2 \frac{E_p}{E_m} \delta_p \quad \ldots\ldots (114)$$

$$f_m = R f_p \quad \ldots\ldots (115)$$

$$E_m = R E_p \quad \ldots\ldots (116)$$

$$f_{jm} = R f_{jp} \quad \ldots\ldots (117)$$

and,

$$b_m = R^2 \frac{E_p}{E_m} b_p \quad \ldots\ldots (118)$$

Scale of Model.—The choice of the scale of the model is entirely arbitrary from the theoretical point of view; but of course it is very much limited by practical considerations. The amount of material in the model, and probably the cost of it, is proportional to the cube of the scale and the area that is covered by the model. Probably also the time necessary to make the tests is proportional to the square of the scale. That would mean that the scale of the model must be chosen as small as possible in order to reduce the cost and time and increase the convenience of the use of the model.

On the other hand, the stresses are proportional to the scale and the deflections to the square of the scale so that the larger models will give better results as far as observations are concerned. Furthermore, if the model is to be a true representation of the prototype the horizontal, and particularly the vertical, construction joints must be represented and that can be done satisfactorily only if the scale of the model is large enough. For the model of the Calderwood Dam it was decided that the smallest scale that could be used was 1:50, and the following considerations are based on a model of such scale.

Modulus of Elasticity of Model Material.—Assuming the modulus of elasticity of the concrete used in the prototype as 3 800 000 lb. per sq. in., from Equation (116), $E_m = \frac{3\,800\,000}{50} = 76\,000$ lb. per sq. in.

Assuming that the maximum deflection of the prototype under consideration will be about 0.5 in., the maximum deflection of the model from Equation (114) will be $\delta_m = \frac{0.5}{50} = 0.01$ in.

The proper observation of the behavior of the model under load will include not only the measurements of the maximum deflections, but the series of measurements at a considerable number of points ranging from $\delta_m = 0$ to $\delta_m = 0.01$ in.

From a practical point of view the deflections that are to be measured at the many points must be kept reasonably large so that the measurements can be made with simple instruments and without the use of complicated and involved measuring devices.

Besides the possibility of measuring the deflections with a sufficient degree of accuracy, the following important point should be stressed: If the model is made of an elastic material and the deflections are relatively large, the secondary influences, such as uncontrollable changes of temperature, inaccuracies in the size of construction joints, displacements of the abutments, etc., have only a small influence on the total deflection, and can be partly accounted for and partly neglected. For stiffer models these secondary influences become more important, and the value of deflection measurements decreases rapidly due to secondary influences as well as to the decreasing degree of accuracy of the measurements.

The requirements of retaining the similarity of the geometrical shape of the dam and model before and after the load is applied, are theoretically correct, but, practically, a considerable increase in the deflections can take place before it could influence the results to a noticeable extent. From these considerations it was deemed advisable to use a material for the model with a much lower modulus of elasticity than that which is required theoretically.

Poisson's Ratio of Model Material.—Poisson's ratio for the concrete used in the prototype is about 0.20 and that for the material of the model should be the same.

If the material of the model has a different Poisson's ratio, then the results of the tests will be influenced in two ways: First, the direct influence that results in redistribution of stresses; and, second, the indirect influence that will manifest itself in a different value of the modulus of elasticity in shear and, consequently, disturb the behavior of the model as far as the shearing deformations and the part of the stresses due to such deformations are concerned.

The direct influence of Poisson's ratio can be taken care of in the model by a proper variation of the size of the vertical construction joints.

An arch dam as well as the model is restrained in the horizontal direction along the axes of the arches, is only partly restrained in the vertical direction due to the weight of the structure that rests on it, and has practically no restraint in the horizontal directions at the up-stream and down-stream faces of the dam. Consequently, Poisson's ratio can influence the stresses and deflections in the horizontal direction along the axes of the horizontal arches,

but it is of no concern in the horizontal direction between the faces and in the vertical direction.

In the horizontal direction the influence of Poisson's ratio will be felt, due to the loads that are applied in the vertical direction and the direct water pressure at the up-stream face of the dam.

Selection of Model Material.—The known arch dam models of the past were built as follows: (*a*) Concrete loaded with mercury; (*b*) celluloid loaded with mercury; and (*c*) rubber loaded with water. None of these meets the similarity conditions with any degree of satisfaction.

The most important discrepancies are the improper ratio of the specific gravities of the loading liquid to the model material and the fact that all these models were built as continuous structures, which could carry considerable tension in all directions.

In selecting the material the first consideration is naturally given to the possibility of making the model out of concrete, the same as the prototype.

Concrete models loaded with water, however, will show very small and uncertain deflections. In order to increase the deflections in the models built in the past, mercury was substituted for water as the loading liquid. This increased the deflections manifold, but at the same time violated an important similarity condition concerning the ratio of the specific gravities of the loading liquid and the material of the model.

The celluloid models loaded with mercury and the rubber models loaded with water have the same drawback. The relationship between the arch and gravity action of the dam is greatly distorted and, consequently, from studies made on such models no proper conclusions can be derived as to the behavior and shape of the dam.

These considerations eliminated the use of concrete, celluloid, or plain rubber as a model material, and a search for a more suitable material was necessary.

For the chosen scale of the model (1:50), and by assuming the maximum range of possible stresses in the prototype to be from 300 lb. per sq. in. tension to 1 000 lb. per sq. in. compression, the corresponding range of stresses in the model material is from 6 lb. per sq. in. tension to 20 lb. per sq. in. compression. In general, this represents a very narrow range of stresses for most of the materials that could be considered.

Besides the requirements that are directly covered by the similarity conditions the material to be selected must satisfy a number of requirements that follow from the similarity conditions indirectly, and that have to be met within the range of stresses in the model.

These are the requirements that characterize the elastic and plastic properties of the model material. The condition of a constant modulus of elasticity must be extended to cover a uniform modulus of elasticity in all directions, which would preclude the use of materials with pronounced directional properties.

On the other hand, the constant modulus of elasticity will preclude materials with pronounced plastic properties that are characterized by a slow recovery

or a high permanent set, and materials that have any considerable skin or internal stresses.

The necessary practical consideration will be to have the material such that it could be brought to the many required irregular shapes without prohibitive expenditure of time and money.

The specifications for the required material based on all previous considerations and in the stress range of the model are summarized in the order of their importance, as follows:

(1) Specific gravity of 2.4.
(2) Uniform elastic properties without any pronounced directional qualities.
(3) Absence of skin or internal stresses.
(4) Low modulus of elasticity.
(5) Reasonably uniform values of the modulus of elasticity at various unit stresses.
(6) Low permanent set.
(7) Low plastic properties.
(8) Properties that will permit the material to be easily furnished in the required irregular shapes.
(9) Poisson's ratio close to 0.20.
(10) Uniform coefficient of thermal expansion.

No material is commercially available or even known that would meet this list of specifications in its entirety, so that the problem resolved itself into the development of a material of the desired qualities. Although the work done to date (1930) in developing such material seems to be rather promising, it showed clearly that considerable additional time must be spent before the ideal material can be found, particularly if the low value of Poisson's ratio is to be maintained.

The practical compromise that was decided on was to drop this requirement and to develop a material that meets the more important demands reasonably close, leaving to the future the development of a material that meets all the specifications. This decision will probably make necessary some corrections in the results of the model test.

A preliminary search disclosed the fact that the combination of the two requirements—specific gravity of 2.4 and low modulus of elasticity—precludes a range of well-known and commercially used materials, most of which have low specific gravities at low moduli of elasticity.

The search made of different, less known materials having cellulose, cellulose acetate, etc., bases, showed that they all have a specific gravity much lower than 2.4 and that the new materials that could be obtained by loading the known materials to the desired specific gravities were unacceptable on account of the high modulus of elasticity and uncertain elastic properties.

Rubber compounds showed the best promise and an extensive co-operative research was made in this field. The main difficulty in this research was that all the customary rubber tests are made at high tensile stresses which approach the ultimate, and very little is known about the general behavior of rubber compositions at low stresses.

Practically nothing is known about the behavior of highly loaded rubber compounds of high specific gravity at low compressive stresses. This research can be summed up briefly as follows:

(*a*) A large number of rubber compounds of the desired specific gravity of 2.4 can be made by using the commercially available fillers.

(*b*) As a general rule, these heavily loaded rubber compounds are unstable and show uncertain elastic and highly developed directional properties at low compressive stresses.

(*c*) The values of Poisson's ratios of different compounds can be varied in rather wide limits; but the tests made, thus far, developed unsteady compounds with directional properties for low values of Poisson's ratio. The customary manufacturing processes seemingly do not give the refinement in control that is necessary in order to obtain material of the same quality by repeated fabrication of compounds of the same composition and manufacturing procedure.

(*d*) From the different compounds it was possible to pick some that, as long as the high values of Poisson's ratio were accepted, have steady and fairly uniform non-directional elastic properties and a low modulus of elasticity, which properties could be satisfactorily repeated.

(*e*) The attempt to find a material that would meet these requirements, having at the same time a low Poisson's ratio led to the conclusion that such material is possible and feasible, but that a considerable amount of research would be required, that would include not only the composition but the manufacturing procedure as well.

Adopting the research made as a basis, it was decided to use for the model a litharge rubber compound of the required specific gravity of 2.4, with a low modulus of elasticity of about 500 lb. per sq. in., and with a high Poisson's ratio approaching 0.5. This compound has uniform and stable elastic properties, and it is only slightly influenced by aging or repeated loading, showing very small differences at a time interval of about six months and at repeated loadings that ran as high as 5 000 lb. per sq. in. hydraulic pressure.

The fabrication of this material can be repeated infinitely, insuring the possibility of manufacturing any desired quantities similar to the original samples; it can be cemented together easily, or cut on a band-saw, insuring the possibility of making blocks of any desired shape and size.

Tension in Horizontal Joints.—In accordance with Equation (116) the ratio of the ultimate unit tension that can be carried over the horizontal joints of the model and prototype must be equal to the scale of the model. A number of difficulties will be encountered in an attempt to fulfill this condition: First, the value of the ultimate unit tension that can be carried over the horizontal joint of the dam is uncertain; and, second, it is rather difficult to provide a joint in the model that would be ruptured at a low tensile stress with any degree of accuracy. The compromise that was reached was to decrease the number of horizontal joints, placing them on the model corresponding to larger lifts than those that were actually used in the prototype.

Foundations of Model.—The proper representation of the foundation conditions of a model is uncertain, due to the fact that practically no information concerning this subject is available.

The practical compromise that was decided upon, was to make the foundations of the model of the same material as the model of the dam, in blocks

of arbitrarily chosen sizes, but with provisions that will make it possible to change them, substituting blocks of different size. At the same time the foundation blocks were partly restrained in the horizontal direction by means of rubber struts, which could again be made of different size and located at different heights. The lower part of Fig. 1 shows this arrangement.

The expectations are that the proper conclusions can be obtained by repeating the tests with foundations of different sizes and with different amounts of restraint. Furthermore, a study of the influence of the elastic deformations of the rock on the behavior of the dam will be made possible.

General Arrangement and Plan of Model.—The model is to be an exact representation of the actual dam constructed to a scale of 1:50, of rubber compound, inside a concrete tank as shown on Fig. 1.

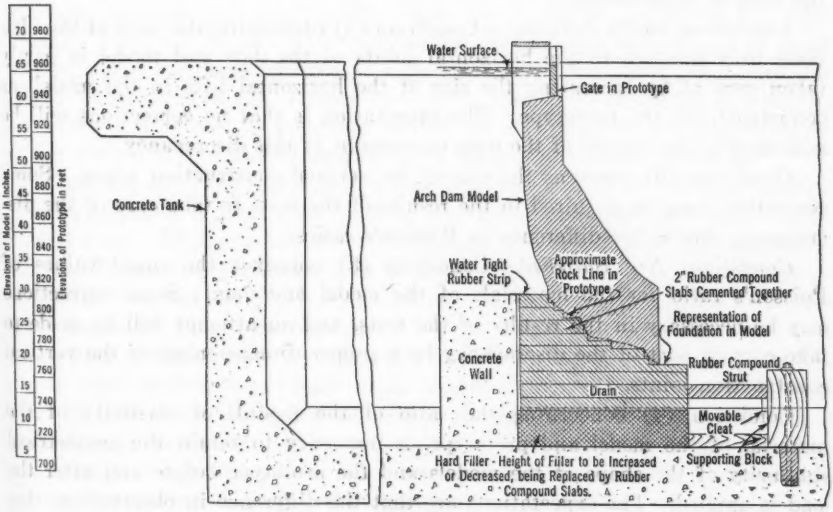

FIG. 1.

Not only will the arch proper find its representation in the model, but the same rubber compound will be used to represent a reasonable part of the abutments and foundations of the actual dam.

The model will be built of separate blocks that are the exact replicas of the concrete blocks of the dam between its vertical joints, but of greater heights than the lifts or distances between the horizontal joints of the actual dam. The necessary modifications in the size of the vertical joints are to be made.

Water is to be used as the loading liquid and deflection and strain measurements are to be made at different points of the dam and are to be repeated for different conditions of foundation deflections.

As far as similarity is concerned, the study of this model can be made by considering the following conditions (see Conditions (1) to (8), previously defined):

(*a*) Condition (1) is assumed to be satisfied.
(*b*) Conditions (2), (7), and (8) are actually satisfied.
(*c*) Conditions (5) and (6) are partly satisfied; and
(*d*) Conditions (3) and (4) are not satisfied.

Conditions Assumed to Be Satisfied.—Condition (1), concerns the assumed constancy of the moduli of elasticity of the dam and model materials.

Conditions Actually Satisfied.—Condition (2) concerning the ratio of the specific gravities of the loading liquids and the material of the dam and model is in this class.

Two other factors in this class are: Condition (7) concerning the representation of the foundations; and Condition (8) concerning the ratio of temperature differences, in model and prototype between the time of building and the time of observation.

Conditions Partly Satisfied.—Condition (5) concerning the ratio of the ultimate unit tensions in the horizontal joints of the dam and model is partly taken care of by increasing the size of the horizontal lifts in the model, as compared with the prototype. The expectation is that no corrections will be necessary in the results of the tests on account of this discrepancy.

Condition (6) concerns the size of the vertical construction joints. Some corrections may be required in the results of the tests to take care of the discrepancy, due to the difference in Poisson's ratios.

Conditions Not Satisfied.—Condition (3) concerns the equal values of Poisson's ratio for the materials of the model and dam. Some corrections may be necessary in the results of the tests, and an attempt will be made to take care of part of the discrepancy by a proper dimensioning of the vertical construction joints.

Condition (4) concerning the ratio of the moduli of elasticity of the material of the model and prototype, is necessary to retain the geometrical similarity of the shape of the models and the prototype before and after the load is applied. The expectations are that the difference in observations due to this discrepancy will be so small that it can be neglected.

Acknowledgments.—This work is a part of the arch dam investigation that is being conducted by the Aluminum Company of America at the request of Edwin S. Fickes, M. Am. Soc. C. E., under the direction of James W. Rickey, M. Am. Soc. C. E., and Mr. J. P. Growdon. The research work in rubber compounds in the Research Laboratory of the Goodyear Tire and Rubber Company was made under the direction of Dr. L. B. Sebrell by Messrs. R. D. Evans and M. J. DeFrance. The Aluminum Research Laboratories of the Aluminum Company of America are assisting in the investigation and, under the direction of R. L. Templin, M. Am. Soc. C. E., R. G. Sturm, Assoc. M. Am. Soc. C. E., and Mr. R. L. Moore, tested the physical properties of the rubber compounds. Messrs. J. P. Den Hartog and J. G. Baker, of the Research Laboratory of the Westinghouse Electric and Manufacturing Company, assisted in formulating the similarity conditions based on the application of the dimensional theory.

JASPER O. DRAFFIN,[14] M. AM. SOC. C. E. (by letter).[14a]—Although the writer cannot add anything to the content of this paper, he wishes to express a feeling of satisfaction that this type of study is receiving attention. To examine the validity of assumptions concerning the principles underlying the use of models is very valuable, because so much hydraulic practice is based upon information obtained from models. More work of this character is needed and the author is to be congratulated for what he has done.

It is impossible to read the accounts[15] of the work done in European laboratories without being impressed by the large amount of space devoted to discussions of models. It is also of interest to note the increasing importance attached to the theory of models.

Undoubtedly, the study of river flow and control in the United States has lagged behind that of Europe, but this condition is changing. The recent floods in New England and in the Mississippi River Valley, added to the Ohio floods of two decades ago, have emphasized the growing importance of flood-control works and have resulted in a public demand for relief from such disasters. This demand has brought responsible opinion to the point of studying flood flow and control by means of models, as evidenced by the establishment of the National Hydraulic Laboratory for the study of these problems.

There are still, however, many people who doubt the possibility of predicting the action of a great river in flood flow by experiments on a small model of that river. If the confidence of the people in the ability of the engineer to solve such gigantic problems is to be retained, all the available theory must be brought to bear upon the design of the models which determine how such great works are to be constructed.

It may not be amiss to call attention to the fact that the early experiments with models were made in a somewhat crude fashion and that the methods have gradually improved. From the early work of Dubuat to that of Reynolds, of Vernon-Harcourt, and, more recently, the work of Mr. Groat, is a long step and great progress has been made in the use of the model as well as a clearer understanding of the significance of the various elements involved in its design and use. Considering the great variety of engineering structures which are built after or tested by models, such as bridges, dams, machinery, ships, and airplanes, as well as structures involving the flow of water, it is desirable that there should be considerable diffusion of knowledge about the necessity of careful design of models and the methods by which it may be achieved. Some experimenters seem unaware of the limitations of their experiments. It is well that the elements of viscosity, temperature, and surface tension should be given prominence by the author since, as Mr. A. C. Chick points out,[16] these factors may be important under small heads or with small openings.

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[14] Asst. Prof., Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

[14a] Received by the Secretary, December 18, 1930.

[15] "Hydraulic Laboratory Practice," by John R. Freeman, Past-President, Am. Soc. C. E.

[16] *Loc. cit.*, pp. 776-777.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FORMULAS FOR RAINFALL INTENSITIES OF LONG DURATION

Discussion

BY MESSRS. W. B. GREGORY, W. W. HORNER, AND ADOLPH F. MEYER

W. B. GREGORY,[1] M. AM. SOC. C. E. (by letter).[1a]—Concerning problems of rainfall and run-off there is often considerable uncertainty in the mind of the designer as to the adequacy of a drainage channel to meet requirements that will represent a reasonable protection against floods. As pointed out by the author the rain that occurs as a maximum once in 50 years or 100 years may occur any time during the period. Furthermore, the rainfall and run-off data are not always at hand, and if available there may be quite a bit of work required to put them into usable form.

The subject of run-off has too often been treated in an empirical manner, with the size of drain designed purely according to the judgment of the engineer in charge or by the limitations set by the amount that may be expended. Even in the latter case it is desirable to know how often the capacity of the drains will be exceeded.

The author has made available information for intelligent design for a large part of the Mississippi Valley and portions outside. He has connected the work of Meyer and Morgan, and has shown that these two authorities agree in basic material. By this means he has given the data on which times of concentration are known in the periods that are of interest in the drainage of areas of considerable magnitude, and thus has rendered a real service to the profession.

The work and expense required to prepare this paper were considerable, and the thanks of the profession are due the author.

W. W. HORNER,[2] M. AM. SOC. C. E. (by letter).[2a]—In developing certain data for his personal need, the author has undoubtedly found himself in a large company of engineers thoroughly appreciative of this particular situa-

NOTE.—The paper by Merrill M. Bernard, M. Am. Soc. C. E., was published in October, 1930, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

[1] Prof., Experimental Eng., Tulane Univ., New Orleans, La.

[1a] Received by the Secretary, November 20, 1930.

[2] Chf. Engr., Sewers and Paving, St. Louis, Mo.

[2a] Received by the Secretary, December 24, 1930.

tion. The writer had occasion several years ago to make a study of rainfall frequencies in the zone just above the 2-hour limit, but under conditions which did not warrant the exhaustive study made by Mr. Bernard. The writer has also been interested in the local study of this same problem at Boston, Mass., as reported[3] by Charles W. Sherman, M. Am. Soc. C. E. There is no doubt that all over the country, as the bigger cities are spreading out and as municipal drainage problems are entering the field of the larger water-sheds, data heretofore considered accurate for design purposes have been found insufficient.

Mr. Bernard has done a remarkably fine piece of work in joining together and interpolating between developed information for short-time rains on the one hand and the "Miami Conservancy" work on the other. It is remarkable, almost surprising, that satisfactory graphs for the 1-day to 4-day frequencies can be made to pass through the 2-hour points. It is also interesting to note Mr. Bernard's belief that no satisfactory diagram for the range above 2 hours can be applicable to the ordinary rainfall curve below the 2-hour duration. In comparison with this, should be noted Mr. Sherman's belief[4] that a single formula can be developed which would cover the entire range up to several days. In a discussion[5] of Mr. Sherman's paper, the writer suggested that a formula devised for Boston did not seem to be satisfactorily in accord with the data for durations in excess of 24 hours.

Mr. Sherman's work in Boston is based upon complete local information covering the full range of duration. In Mr. Bernard's studies no actual data are available for the duration period between 2 and 24 hours, and his results are of value because of the remarkable concurrence at the two edges of this zone. It supplies a new tool for general use in this intermediate field, a working instrument apparently much more reliable than anything that has been published before. There is no doubt that to municipal engineers as well as land drainage engineers the results of Mr. Bernard's work will be exceedingly helpful. It can be used to good advantage until the time comes when some project of the general magnitude of the Miami Conservancy work may justify the expense of determining frequencies in this intermediate zone directly from recorded rainfall information. When this time comes, available authoritative studies will be available throughout the entire range, and it will be possible to determine whether any single formula will be found to be completely satisfactory. In the meantime, Mr. Bernard's work should receive the appreciation of all practicing engineers interested in storm-water drainage.

Adolph F. Meyer,[6] M. Am. Soc. C. E. (by letter).[6a]—It is always a pleasure to read a paper like that of Mr. Bernard which sets forth the results of an extensive study in a clean-cut manner, ready for practical application.

Of the fundamental principles referred to by the author, the writer desires to emphasize the following, expressed in his own language:

[3] *Proceedings*, Am. Soc. C. E., April, 1930, Papers and Discussions, p. 717.
[4] *Loc. cit.*, p. 720.
[5] *Loc. cit.*, September, 1930, Papers and Discussions, p. 1746.
[6] Cons. Hydr. Engr., Minneapolis, Minn.
[6a] Received by the Secretary, December 29, 1930.

1.—From the viewpoint of most structures involved, the effect of actual rainstorms, varying in intensity as they do, is practically the same as if the precipitation had been uniformly distributed over the given time interval.

2.—The sum of the records of several stations within a limited area, even if of considerable extent, gives a better indication of the probable rainfall experience of a given locality than the record of a single station in that locality. Such combined records are reasonably equivalent to a long-time record for that station.

The writer hopes that these two fundamental principles, supported as they are by ample proof in the literature of hydrology, will soon receive general acceptance by the profession. Such acceptance will be in the interest of both safe and economical structural design.

Another factor in design, which the author rightfully emphasizes, but which is often neglected or cloaked in generalities, is the element of probable average frequency of recurrence of given phenomena. Although at best a prediction can only represent a probable average experience, that is no reason why the best possible prediction should not be made in reasonably specific terms, and then these terms should be used as the basis for design.

In his opening discussion, the author concludes that the relation between rainfall intensity and duration is more accurately expressed by the parabolic formula, $i = \frac{C}{t^n}$, than by the formula, $i = \frac{a}{t + b}$. This conclusion is apparently in conflict with what follows in the paper. The author specifically points out (under the heading "Intensity Formula Types") that the Meyer formulas of the type, $i = \frac{a}{t + b}$, give correct results over time intervals to about 200 min. They are intended for application only to time intervals up to 120 min. Fig. 1 shows that Mr. Bernard's parabolic type of formula can be used only for time intervals longer than 40 min. The writer has never suggested the application of his formulas to time intervals longer than 120 min.

At best, all formulas of rainfall intensity are empirical. Since they are derived from a given group of physical data, it follows as an elementary principle that they cannot be extended, with the slightest assurance of accuracy, more than a few per cent. beyond the limits of the data on which they are based. Discussions tending to show that certain formulas cannot be applied to data lying far outside the limits of the group of base data used in their derivation, are beside the mark. All too frequent attempts are being made to extend formulas and principles far beyond the scope of the base data. When such attempts lead to error, the formula or the principle often is blamed. The author, however, specifically points out the limitations of the several formulas.

Natural phenomena such as rainfall intensity and duration are virtually haphazard occurrences. The derived formulas are empirical, and since they do not embody a fundamental physical relationship, such formulas are necessarily limited to the base data from which they are derived.

The writer would like to see the author's conclusion respecting the limits of application of his charts added to the title of each chart lest these charts be reprinted and used without the author's qualifying statement that they are intended for application to duration periods greater than two hours.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

THE BUTTRESSED DAM OF UNIFORM STRENGTH

Discussion

BY MESSRS. B. F. JAKOBSEN, AND CALVIN V. DAVIS

B. F. JAKOBSEN,[3] M. AM. SOC. C. E. (by letter).[3a]—The author presents a design method for a multiple-arch dam buttress, which should result in a structure of uniform strength. In general, at least, it should be possible to eliminate the possibility of buckling by providing counterforts, and since in Mr. Schorer's design two principal stresses are zero, the requirement of uniform strength is met when the third principal stress is constant throughout the buttress. As the author shows, this leads to a buttress having a horizontal section of variable width, as in Fig. 2, instead of a constant thickness, as has been customary in the past. The writer has calculated[4] the stresses in a buttress having a rectangular section in a horizontal plane. The section, for which the stresses were computed was an inclined, nearly normal section. The result shows, that the maximum principal stress (compression) was nearly constant, varying from about 260 to 286 lb. per sq. in. The intermediate principal stress, at right angles to the plane of the buttress, was zero, and the least principal stress (tension), varied from zero at the down-stream face to 84 lb. per sq. in. at the groin. This means that the factor of safety in compression is ten or more, while in tension it is three or less, unless steel is provided to take up the tension. This indicates the superiority of the author's design method.

The elimination of tension can presumably be accomplished by lengthening the buttress, but since stress calculations are complicated and time-consuming, it is difficult, if not practically impossible, to arrive at an economical design. Mr. Schorer's design method is therefore of great practical value.

In connection with the design of a fairly high multiple-arch dam the writer has had an opportunity to compare Mr. Schorer's method of design with the customary one. The consulting engineer, who prepared the design, first submitted one of the usual type having a constant thickness in a horizontal plane. This design had a small amount of tension, and it was decided to apply

NOTE.—The paper by Herman Schorer, Assoc. M. Am. Soc. C. E., was published in November, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings* as follows: January, 1930, by Messrs. James Girand and Fred A. Noetzli.

[3] Cons. Engr., Los Angeles, Calif.

[3a] Received by the Secretary, December 19, 1930.

[4] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 306.

Mr. Schorer's method. The result was highly satisfactory to both parties, and a design with a tapered buttress in a horizontal section was adopted.

In a paper entitled "Stresses in Multiple-Arch Dams"[5] the writer treated the arch as monolithic with the buttress, as the author has done in Fig. 5. This treatment gives a shorter buttress than the assumption that the deck transmits load to the buttress only in a direction normal to the groin, as in Fig. 4. This latter involves the assumption of a frictionless joint between the deck and the buttress, and the writer questions whether such a joint is a practical possibility, considering that it must function for an almost unlimited time and under any conditions. These considerations would lead to the adoption of the arrangement shown in Fig. 4, as the safer design assumption, if it were not for the possibility of an upward thrust of the deck, due to water-soaking, combined with drying out of the buttress, and the consequent shrinkage. This tendency will be more pronounced when the temperature of the air is less than that of the water.

Most multiple-arch dams, of which the writer has knowledge, show cracks in the buttresses which, near the groin, are normal to the direction of the groin.[6] These cracks may be due to shrinkage and swelling, as just outlined, or they may be due to tensile load stresses. In some of these dams the writer inclines to the opinion, that the cracks are due primarily to shrinkage and swelling. If the deck could be constructed of segments, say, 10 ft. high, which were provided with open joints above and below, so that these segments could not interact lengthwise, the condition illustrated by Fig. 5 would exist. This might involve difficulties for the multiple-arch type, but could easily be accomplished for the flat deck type of dam. As multiple-arch dams are generally built, it seems to the writer, that some swelling of the deck combined with shrinkage of the buttress must be considered as a possible, and probable, state of affairs and the designer must be governed accordingly.

The author states that the writer advised contraction joints in buttresses and arches in 1924. This is not quite correct, since the writer advised consideration of "temporary" joints[7] and suitable construction methods.

Recently, in connection with the design of two multiple-arch dams, the writer, in collaboration with other engineers, gave this question careful consideration. The consensus of opinion was, that it was better to reinforce the buttresses in an attempt to prevent unsightly cracks, than to attempt to provide shrinkage joints, which would be likely to promote cracking in the general direction of the joint, unless they started at the deck and went entirely through to the foundation (see Fig. 11). Additional steel was placed near the groin line and parallel to it, in order to counteract the swelling-shrinkage effect already mentioned. In addition, it was agreed that the buttresses should be constructed as far ahead of the arches as possible, in order to give them a chance to shrink before the arches were poured.

This paper will have an important and beneficial influence on multiple-arch dam design, and the author is to be congratulated for presenting it.

[5] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 276.
[6] *Loc. cit.*, p. 311; see Fig. 42, showing cracks in Lake Hodges Dam.
[7] *Loc. cit.*, p. 311, last paragraph.

CALVIN V. DAVIS,[8] ASSOC. M. AM. SOC. C. E. (by letter).[8a]—The author has made a valuable contribution to the art of dam design. This paper emphasizes the important principle that a dam is a series of elementary curved, inclined columns, each of which transmits part of the water pressure from the water-bearing surface through the structure to the foundation. This discussion will treat several phases of this principle, as developed by Mr. Schorer, and will also demonstrate that the same principle may be developed by the trial method of design.

In his "Synopsis", the author makes the following statement:

"The theory is based on the conception of elementary arched column units which transmit the water pressure from their respective deck areas directly and in uniform compression to the ground, together with the accumulated dead load. The elementary column units can be combined to a monolithic buttress system in which the first principal stress is uniform compression throughout and the second principal stress is zero."

As the writer understands this, Mr. Schorer considers that if each of a series of inclined elementary columns were in equilibrium under their respective loading this same condition would apply when the columns were combined to form a monolithic buttress. In the writer's opinion the stress conditions in a buttress considered as a monolith will not be similar to the stress conditions in the elementary columns when they are considered independently. The elastic properties of each elementary column will have some relation to the elastic properties of each adjacent column and, consequently, in arriving at the stresses in the structure, the deformations in the buttress as a whole must be considered.

Buttresses are not ordinarily constructed as rectangular sections and their structural properties are materially influenced by combining haunch or monolithic deck areas with the buttress section. For instance, the monolithic deck shown in Mr. Schorer's design example (Fig. 11) will have the effect of moving the center of gravity of a horizontal buttress section in an up-stream direction. When the buttress is considered as a monolith the effect of this added up-stream area will be to increase the eccentricity of the resultant loading. If the inclined columns were designed so as to show zero second principal stress along the joints when considered independently and neglecting the monolithic action of the deck, the second principal stresses would be in tension down stream from the deck if the buttress were analyzed monolithically and the effect of the horizontal deck area were included in the computations. In the monolithic analysis the increased eccentricity would lower the vertical normal stresses in the up-stream portion of the buttress and this, in turn, would change the second principal stresses from a value of zero down stream of the deck to some value in tension. On the other hand, the second principal stress throughout the deck will be compression. The writer believes that the results obtained by considering the buttress as a monolith will be less favorable than those obtained by Mr. Schorer's method and, therefore, would be more conservative to use in buttress dam design.

[8] Chf. Designer, Ambursen Dam Co., New York, N. Y.

[8a] Received by the Secretary, December 20, 1930.

The writer is in thorough agreement with Mr. Schorer in the statement that the greatest economy may be obtained through the use of a buttress dam in which the first principal stresses are of uniform intensity and approach the allowable value throughout the structure. The results of several years of research, however, have convinced the writer that the same structural properties, that is, uniform strength, zero second principal stress, etc., may be obtained in a buttress of triangular shape with approximately rectangular horizontal sections. These properties may be readily controlled through the proper selection of both the up-stream and down-stream buttress slopes and also through the use of pilasters on the down-stream face of the dam. The second principal stresses in the designs developed from the research already mentioned were relatively high values in compression and this property, in the writer's opinion, adds a substantial margin of safety to the structure. Failure to include the effect of possible second principal stress in compression or tension is perhaps the greatest objection to the use of Mr. Schorer's method, which gives results only with reservoir full. Under all other loadings the second principal stresses will be compression. The inclined joints will only be on trajectories of first principal stress with reservoir full, and with the reservoir partly filled light shearing stresses will be developed across these joints. The writer believes it is important to determine these stresses and to provide keys in the joints or other means to take care of them in order that the monolithic character of the structure as a whole may be preserved.

For the reasons outlined the writer prefers the trial method of design, even if its use is somewhat tedious and involves considerably more labor than Mr. Schorer's method. The trial method is general and may be applied to the analysis of any type of dam. It is understood, of course, that it must be carried far enough to determine completely the principal stresses and their directions and also the shearing stresses in all parts of the structure. It is no longer adequate, as Mr. Schorer points out, to base a design on the determination of vertical normal stresses alone, as the second principal stresses may have high values in tension with the trapezoid of vertical normal stress entirely in compression.

In applying the trial method the principal stresses and shearing stresses may be determined directly from the vertical normal stresses by means of successive integrations. The writer believes that this procedure was first introduced in the United States by the late William Cain, M. Am. Soc. C. E., in 1909.[9] B. F. Jacobsen, M. Am. Soc. C. E., first showed the necessity for taking into consideration the principal stresses in a multiple-arch dam.[10]

Figs. 18 to 23 illustrate the application of the trial method of analysis to the design of a typical high dam of the gravity type; and at the same time they illustrate the development of the inclined column principle. The computations were not carried out for the upper half of the structure, but were carried far enough to illustrate the points raised in this discussion. No uplift pressure was assumed in making these computations. It is essential, of course, to

[9] *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208.
[10] *Loc. cit.*, Vol. LXXXVII (1924), p. 276.

consider uplift pressure in gravity dam design. However, these pressures are more important in considering sliding and overturning than stress intensity and distribution. It is possible that uplift pressures may vary from zero to

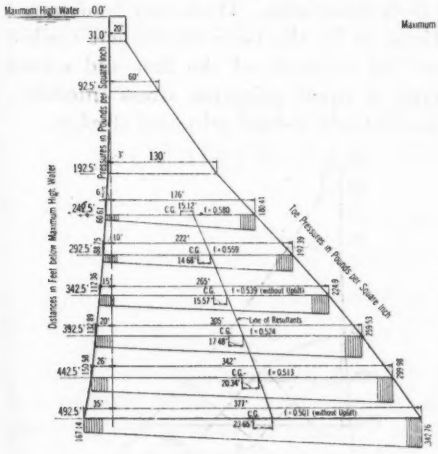
Maximum High Water
Distances in Feet below Maximum High Water
Pressures in Pounds per Square Inch
1st Pressures in Pounds per Square Inch
Line of Resultants

FIG. 18.

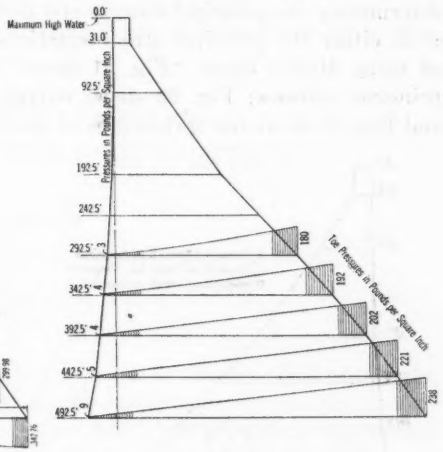

FIG. 19.

100% of the full static head at the heel and, of course, any assumption of this kind may give fictitious values. Inasmuch as the principal stresses may have maximum values with no uplift pressures, this assumption was used.

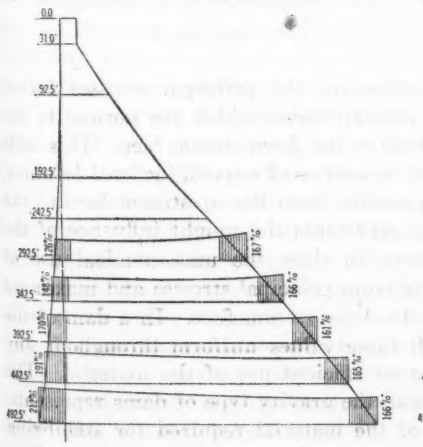

FIG. 20.—HORIZONTAL NORMAL STRESSES.

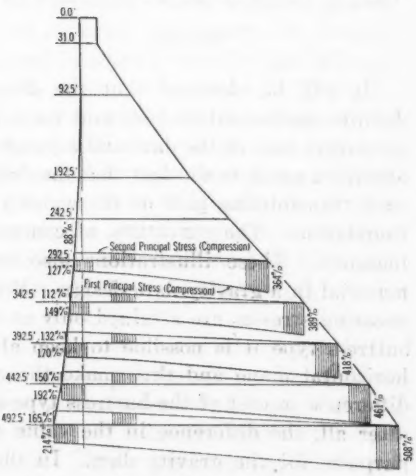

FIG. 21.—PRINCIPAL STRESSES.

Fig. 18 shows the vertical pressures on horizontal planes as obtained by the well-known law of the trapezoid. By taking the algebraic sum of all vertical stresses acting on elementary prisms, shearing forces on these prisms may be obtained, and from these may be obtained shearing stress intensities, as shown

on Fig. 19. The same procedure may be followed in determining horizontal forces on elementary prisms and from these may be obtained the horizontal normal stresses shown on Fig. 20. The necessary elements are now at hand for determining the principal stresses and their directions. These may be obtained from either the principal stress equations, or by the more convenient method of using Mohr's circle. Fig. 21 shows the intensity of the first and second principal stresses; Fig. 22 shows curves of equal principal stress intensity; and Fig. 23 shows the trajectories of the first and second principal stresses.

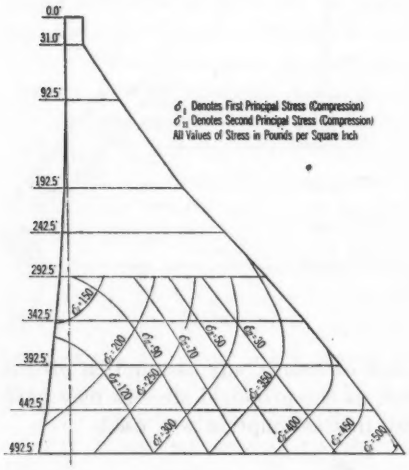

FIG. 22.

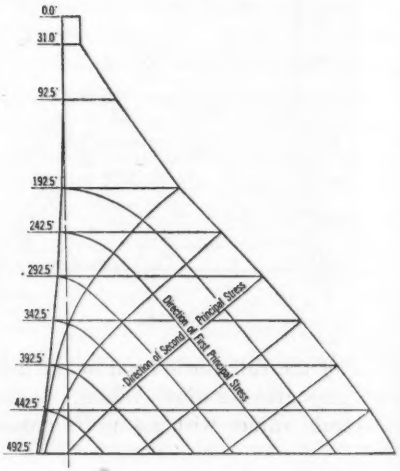

FIG. 23.

It will be observed that the directions of the principal stresses follow definite mathematical laws and form regular curves which are normal to the up-stream face of the dam and asymptotic to the down-stream face. This calls attention again to the fact that the dam is a series of curved, inclined columns, each transmitting part of the water pressure from the up-stream face to the foundation. The curvature, of course, represents the weight influence of the masonry. These illustrations also serve to show the uneconomical use of material in a gravity dam design. Maximum principal stresses and maximum shearing stresses are attained only at the down-stream face. In a dam of the buttress type it is possible to keep all these values uniform throughout any horizontal plane and thus make the most efficient use of the material. The difference in cost of the buttress type and the gravity type of dams represents, after all, the difference in the value of the material required for stabilizing purposes for the gravity dam. In the gravity dam dependence is placed on the weight of material in the structure alone to attain stability. This means an unsatisfactory stress distribution. In the buttress type, stability is obtained through the use of an inclined up-stream face which introduces heavy stabilizing water loads on the structure. This allows an economical use of material in the structure and makes possible the attainment of satisfactory stress distribution.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

ECONOMICAL DESIGN OF HYDRAULIC PIPE-LINE DREDGE

Discussion

BY R. L. VAUGHN, ASSOC. M. AM. SOC. C. E.

R. L. VAUGHN,[1] ASSOC. M. AM. SOC. C. E. (by letter).[1a]—This paper, the most valuable which the writer has seen on the subject, is, perhaps, unique. It points the way to what has seemed a hopeless task, the application of scientific reasoning to that exceedingly complicated affair—a pipe-line dredger.

The writer has made several attempts to apply hydraulic theory to the dredger pump, and invariably has come to an abrupt stopping place at Equation (2). This is the customary formula given by many authorities on pump design, but the writer always has doubted whether it was even approximately true when applied to the pump on a pipe-line dredger. If the material passing through the pump were a uniform one, such as water or mercury, the formula would be unquestionable, but the mechanical mixture generally handled may present an entirely different set of problems.

The reason for introducing the factor, w, is to convert the fundamental relation, "power equals force times distance divided by time", into a form which can be applied readily to the conditions of a centrifugal pump. If the pump is handling a mechanical mixture the weight of which may (and often does) vary widely from moment to moment, the question arises as to what value should be assigned to w. In the ordinary texts, w is used as if it were the weight of material actually passing through the pump, although several observations indicate that this may not be correct.

Indicating watt-meters have frequently been observed while connected to pump motors of constant speed working on long pipe lines. When the digging was stopped and clear water pumped, there was no observed change in power, except such as could be accounted for by change in weight lifted to the pump, until such time as the line began to clear. If the ordinary equations are correct, there should have been a marked change in power as soon as clear water reached the pump.

NOTE.—The paper by John Francis Cushing, M. Am. Soc. C. E., was published in November, 1930, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[1] Cons. Engr., San Francisco, Calif.

[1a] Received by the Secretary, December 16, 1930.

Not infrequently some inspired enthusiast, with a background of theory on the design of centrifugal pumps, comes forth with a proposal to revolutionize the dredging business by arranging to introduce the material into the water after it has passed the pump. Mechanical details vary with the originality of the inventor, but the basic idea is that the pump will be handling clear water and can be designed to give much higher efficiency than an ordinary dredger pump will show. Then too, of course, the added advantage of greatly decreased wear on the pump is stressed.

If any advantage were to be gained by keeping the value of w low, that is, if the weight of the material actually passing through the pump were related to the question, there would be a great possible saving in power by keeping it low in the equation for power, and the idea might not seem so far-fetched.

A number of hopper dredgers have been built with arrangements for pumping the load ashore. The general arrangement has been a long suction line passing under the hoppers, with valves in each hopper discharging into the pipe. The material was dumped into the suction because this was the obvious and simple way to do it. If any real advantage existed in keeping the value of w low while the mixture passed the pump, it would be comparatively easy, and not too expensive, to insert the material into the discharge line instead of into the suction. No one seems to have discovered any such advantage.

Designers have assured the writer that, if a pump were handling clean water and a boulder were introduced into the line, there would be a momentary fluctuation in the watt-meter, corresponding to the change in weight of the mixture, while the boulder passed through the pump. The absurdity of this contention will become evident if two extremes are considered. Suppose that a pump is pumping water against a considerable lift and friction head. Imagine that the liquid (in the pump only) is replaced momentarily by mercury. According to the theory the power input should increase at once about eightfold. Again, suppose the water is replaced by some incompressible but weightless liquid. Then the power should drop to zero. Evidently, in either hypothetical case, the total work being done on the system would remain unchanged, except for internal pump losses.

For these reasons, when considering a dredger pump, the writer has always felt reluctant to accord to Equation (2) that veneration and respect which the authoritative elements of a subject are supposed to merit. Furthermore, the texts then proceed to develop the equations for the pump by inserting a large number of constants, for which a value is never given, consequently the writer's opinion of the texts with which he is familiar, is not high. Evidently, the power consumed by a dredger pump is not wholly independent of the weight actually passing through it; but it seems probable that the relation is a complicated one, involving, among other things, perhaps, the characteristics of the actual pump in each case. It will be appreciated if the author will offer any explanation which will make this question clearer.

If it is understood that w, as used in the paper, is the average weight of mixture handled, and that this average remains sensibly constant, then the foregoing objections do not apply to the mathematical treatment given. The assumption is not true in practice because as a rule a dredger does not dig on

the return swing, and never digs when setting ahead; in fact, it often happens that the line is entirely cleared of material while "fleeting ahead".

Equation (4) expresses the condition that the friction head varies directly as the first power of the length of pipe line. This, for some kinds of clay, is far from true. If clay is pumped 3 000 or 4 000 ft., the greater portion of it will be discharged on to the fill in the form of lumps and balls, while if it is pumped 10 000 or 12 000 ft. (even without a booster), in many cases a large portion will come out as mud. Therefore, to state that the friction head is a linear function of the length of line is to state that, for a mixture of a given weight, the friction is the same whether the mix is a thin mud or a combination of water and balls. Manifestly, this is not true, and yet Equation (4) is part of the basis for Equation (10).

There appears to be some confusion in the meaning of C. From the notation, and in the text under the heading "Economic Velocity under Typical Service Conditions", it would appear that C is the unit cost for labor plus the unit cost for all power consumed on the dredger. In Equations (1) to (10), it appears that only the unit cost of power consumed by the main pump is included. Experience indicates that, generally, when digging clay, at least one-quarter as much power will be required by the cutter as is used by the pump, and the ratio is often higher. This would seem to be too large a factor to be neglected, even in an approximate solution.

Nothing is more certain than that w will vary with the velocity in the pipe line, and yet it seems to be treated as a constant when the equation is differentiated. Thus, the writer looks upon the mathematical treatment resulting in Equation (10) with some misgivings and would like much more explanation of it. However, it is interesting, and worthy of careful study, as it may indicate correctly the effect of varying certain factors in the general problem.

Moreover, it must be confessed that the results which the author gets by using Equation (9) are in line with the writer's experience. A decrease in weight of material gives a relatively great increase in output. A considerable variation in pipe-line diameter does not produce much change in desirable pipe-line velocity. Again, on the average, large pipe lines, with suitable increases in power, make for reduced unit costs, and many contractors are obtaining larger machines.

In general, the paper describes a fine dredger which affords little to criticize or question in regard to its mechanical features. The need for strength and rigidity in the hull is illustrated by the experience of the dredger, *Houston*, which sank in a depth of 30 ft. of water in Galveston Bay during the hurricane in 1915. At the time she sank both spuds and the ladder were down and the hull came to rest nearly level longitudinally, upside down, with the bottom making an angle of 55° with the horizontal. The hull was of wood and was exceptionally well designed and well built. It was 165 ft. long, 44 ft. beam, and 14 ft. deep, with no trusses extending above the deck. The stresses which this hull underwent in rolling over as it sank, then being rolled back and raised, can be imagined. (In explanation of how a hull 44 ft. wide could roll over in 30 ft. of water, it may be well to state that the hurricane had raised the water to a depth of about 44 ft.) Rolling the hull back again after the

storm, when there was only 30 ft. of water, was difficult. Extensive re-caulking and some repairs around the ladder and spud-wells put the dredger back in service, and, with some additional repairs, the hull is in good condition to-day (1931).

The installation of a gyroscope compass is a refinement not hitherto considered justified on a dredger, although as early as 1913 a device was proposed, consisting of a recording mechanism actuated by a gyroscope and also connected to the ladder, so that the device would draw a cross-section of the cut at each set-up. This proposal was rejected on the ground of expense, and for the further reason that it required the best energies of the organization to keep the dredgers running, without adding further complications.

Some mechanically successful Diesel-electric dredgers have been built. They are not without advantages. Practical Diesel speeds do not correspond to suitable dredger-pump speeds, and if direct-connected, it is necessary to use gears. The advantages of flexibility to be had with direct current are recognized, while the shocks which a Diesel engine gets when directly geared to a cutter are not conducive to trouble-free operations. However, when the added capital cost, the additional skilled labor required, and the additional complications are all considered, the writer has never been fully convinced that the Diesel-electric is the best design. On a large machine it probably is the best, due to the possibility of using several moderate sized duplicate main engines. Up to a size of at least 1 000 h.p. on the pump, the direct Diesel design seems to be satisfactory.

No information is at hand concerning the power distribution conditions throughout the territory in which this dredger is to be used. On the Pacific Coast power is abundant, cheap, and available nearly everywhere at 11 000 volts, alternating current. As a result, most successful dredgers there are electric ones. These are simple, relatively cheap to build, and inexpensive to operate. Power is brought to the dredger on pole lines (often built over the water) and by 1 000 ft. or so of armored cable; it is then reduced to 2 200 volts, the transformers being located on the dredger or on a scow moored alongside. All the principal motors are usually 2 200-volt alternating-current machines. Any other voltage is readily available through small auxiliary transformers.

Rotary converters could be installed and the power converted to direct current on the dredger, but this has never been done. Direct current has advantages as to flexibility, but these have not been considered sufficient to warrant the additional investment, complications, and power loss. The main motors are of the phase-wound induction type. Some variation in speed is obtained by means of variable resistances in the rotor circuits (external). However, variable pumping conditions are usually met by changing the diameter of the pump runner.

A recently developed device will give these motors a range of speed from 25% below to 25% above the rated speed, maintaining constant torque, with little loss of energy. The device has not yet been applied to a dredger as far as the writer knows, but it is being considered for that purpose.

The first turbine dredger to be built had a turbo-generating unit producing alternating current at 2 200 volts, and was otherwise arranged as an ordinary electric machine. It was a simple matter to install transformers on a scow, bring power from shore when available, and dispense with the steam plant. It was found that operating costs could be much reduced in this way, and the dredger is still so operated. At present (1931), larger motors have been installed and the steam plant could not begin to give the output which can be had by taking power from shore.

On paper, it is easily possible to show that power can be generated on board more cheaply than it can be bought ashore. These calculations, when applied to a dredger, usually prove to be "a delusion and a snare". Dredging contractors find it advantageous to abstain from the power business whenever possible.

It is understood that the anticipated conditions were such that the *New Jersey* must have her own power plant, and that, when considering the design, the advantages of direct current were attractive; but it would be interesting to know whether the possibility of obtaining power from shore, occasionally, was fully considered when selecting the type of electrical equipment which was installed.

The author's statement that the largest practical diameter of pipe is the most economical, may be misleading. It is true as regards power expenditure per unit of material transported. It is far from being true as regards a dredger of stated power. For a given machine, when pumping mud, the largest line within reason will be the best, but in pumping any mixture that will segregate, there will be a minimum velocity which must be maintained. This velocity will vary with the size of the particles in the mixture, and with the percentage of dilution. If the line is too large, a much diluted mixture must be pumped, and there is no economy in that. In fact, most dredgers do not have power enough for the size of the line; very few of them can pump all the clay they can dig for a distance of 6 000 ft. It is often a question of whether or not more power should be added. More power means more pressure, which causes trouble. The ball joints freeze; there is more trouble in holding the line together; and heavier pipes are required. Often, it is better to put in a booster.

Selecting an economical size of dredger for a contractor involves so many practical considerations, that a mathematical analysis can only be used as a guide to the judgment on a few factors. The tendency is toward larger sizes, undoubtedly. The author has selected a 30-in. machine. For jobs of some magnitude this is probably not too large, although for a series of scattered jobs, each of, say, from 100 000 to 500 000 cu. yd., coming up from time to time, some contractor with a well-designed, 22-in. machine, having a 500-h.p. cutter and a 1 500-h.p. pump, could furnish plenty of competition.

Be that as it may, the 30-in. diameter having been decided upon, the next step is the question of how much pump power is suitable for this size of line. This may be approximated by remembering how much power the designer wishes he had had on some previous dredger which worked fairly well, and estimating that the powers should vary, roughly, as the squares of the respective pipe-line diameters. The next step is to estimate how much digging power is

needed to keep this pumping outfit busy on a reasonably short line. Experience indicates that at least one-quarter the pump power should be available for the cutter, and more may be needed. The *New Jersey* has a nominal ratio of one-fifth which may be found ample, particularly as this is an intermittent service and the drive will stand a generous overload.

The writer fully shares Mr. Cushing's opinion concerning clay with boulders in it. Any dredger which will perform, even passably well, in such material is a good machine. During the early days of the dredging business it was considered necessary to install a "stone box" in the suction line just ahead of the pump. This device was provided with grate bars intended to prevent the entrance into the pump of anything that could not pass through. Eventually, it was found that the trouble with the stone box was that it worked too well; the delays caused by cleaning it were intolerable. Runners were obtained which would pass anything that could come up the suction line; the gain in running time far more than made up for any loss in pumping efficiency. Present practice is to have a suction larger than the discharge, in which case it is not necessary to have a runner which will pass larger objects than can pass down the line. With a cutter having a maximum opening of only six-tenths the diameter of the pipe there should be no rocks sticking in the pump, although trash may. Other cutters are used, having larger openings. In such cases, where necessary, the condition is met by inserting intermediate rings in the cutter, or by placing a bar across the mouth of the suction pipe.

An extended experience with trash, buried forests, saw-mill slabs, pile stubs, etc., might modify the author's opinion concerning the relative value of high efficiency in a dredger pump. Large rocks can be excluded from the suction, but these other things can not, and the manner in which they can plug up a runner is truly remarkable. Ordinarily, if the runner can pass anything which can pass through the line, there will not be much trouble from this source. Runners with smaller openings can often be used, and are better. A few years ago a two-vane runner was developed at New Orleans, La., for pumping sewage. Built somewhat after the fashion of a weedless propeller, it would pass just about anything, and when adapted to a dredger it did yeoman service in the swamps. The output of the dredger increased from 1 600 to 10 000 cu. yd. per day, and the operators spent little time in vain regrets over the sacrifice in mechanical efficiency.

The author uses an efficiency of 70% for his pump, which is to perform against heads ranging from 125 to 220 ft., with no alteration other than a variation in speed. Few centrifugal pump designers would undertake to design a water-pump to maintain such an efficiency over this range. When it is remembered that in a dredger pump the disk friction is much higher, that the turbulence is necessarily far greater, that the passages through the pump must be much larger than is desirable, that the surfaces of these passages are anything but smooth and highly polished, and that wear is constantly changing the theoretical lines, it is very doubtful whether any dredger pump even approaches this efficiency. If the author has any data to sustain such an efficiency, such information would be interesting.

Table 2 has been computed using one set of values for w, f, b, and p, and variable values of L. It is certain that w will vary with the length of line and with the velocity, while, for some materials, f will vary with the length of line independently of any variation in w. When this last factor varies, then f, b, and p are all variables. It would be a contribution of the greatest value if Mr. Cushing could give any experimentally determined values for e, f, n, and p.

The author states that the volume of a clay will be a linear function of the water content, and, of course, if this is true, then the statement follows, as given, that in a fill where the change in dimension is all in one direction the settlement will be in greater ratio than the change in water content. If there are any observations to substantiate this relation between water content and volume they would be of value. Experiments on the bulking of sand give different results. Anything approaching accuracy in estimating the volume of a cut, in clay or mud, required to make a given fill, is very difficult, and contractors as a rule are reluctant to bid on fill measurements.

A combination of causes probably contribute to the great difference in shrinkage between the first and second fill shown in Table 1. For one thing, the description indicates that the first fill did not come much above the water level. Under such circumstances a comparatively light load is placed on the original bottom, and there is little chance for the newly deposited fill to dry out. Furthermore, with less than two-thirds of the fill placed, a large settling basin must have remained and, consequently, there was only a small loss through the spillway. When the second fill was placed, it was mostly above water where it could dry out and shrink; also, the superimposed load expelled water from the first fill and from the underlying bottom. Again, during the second season's work the settling basin was becoming filled, and there were greater losses through the spillway. Under some circumstances these losses become quite appreciable, and no computations of volume ratio are of much value unless they are taken into account.

The description of liners in the hull pipe and fittings brings to mind an experiment made by the writer, which may be of interest. During the World War the problem of supplies and repairs was especially difficult on the Pacific Coast and any desperate expedient was justified. One job on hand involved the moving of about 2 000 000 cu. yd. of mud and 750 000 cu. yd. of clay which was in a fairly level strata under a portion of the mud. The mud was a fine slippery bay silt, containing less than 1% of heavier particles, grains of sand, and small pieces of broken shell. The clay contained considerable small sharp gravel. This last combination works on the principle of the diamond drill bit and is guaranteed to wear out anything rapidly, but there was no reason to anticipate more than nominal abrasion from the mud. It was proposed to strip as much of the mud as possible and deposit it in one fill and then dredge the clay and deposit it in another, and closer, fill.

Pipe was scarce, exorbitant in price, and was only available in low carbon steel which wore out rapidly. Records were on file showing cases in which at least 2 000 000 cu. yd. of sand had been handled through a single installation of wooden stave pipe. Several manufacturers of this pipe were located in the

the vicinity, and were in a position to fill orders promptly. If wooden pipe would handle 2 000 000 cu. yd. of sand, there was no reason to think it would not give equally good service with soft slippery mud. Of this pipe, 1 000 ft. was purchased and installed in the most permanent part of the line. It lasted just five days during which it carried about 50 000 cu. yd. of mud and then blew up. More steel pipe was coming through the shops but there was none to spare on hand, and the experience was aggravating to say the least. No clay at all had been pumped.

What had happened was that the coarser particles settled to the bottom of the pipe where they were dragged along by the stream of mud and did a maximum amount of cutting. A width of about 6 in. on the bottom of the pipe was practically all cut away while the remainder of the pipe showed no wear at all. This circumstance suggested another experiment. Another 500 ft. of wooden pipe was purchased and sent to the pipe shop where the bottom third was lined with soft steel plates $\frac{1}{8}$ in. thick. All the liners were punched to template and held in place with small bolts, the purpose being that when the plates wore out they could be easily replaced. This experimental pipe was installed in the line and it never did wear out. It carried about 1 000 000 cu. yd. of mud and 750 000 cu. yd. of clay, and outlasted two sets of steel pipe.

The method used by the author to determine velocity in the pipe line is interesting, but can be used only in fresh water. Permanganate and several different dyes have been used with satisfaction in salt water. The coloring matter is easily introduced into the suction. A rough rule which is nearly correct is as follows: The velocity in the line, in feet per second, is equal to four times the horizontal distance, in feet, traveled by the stream while it falls the first foot after leaving the end of the pipe. That the rule may be correct, it is necessary that the pipe be full at the end, that it be discharging horizontally, and that the stream be falling freely.

A swivel joint near the lower end of a steel spud has often been discussed, but this is the first time the writer has heard of its actual use. While it is true that there is comparatively small stress on a spud near its point, still this sounds like an additional complication which may give trouble. Unless they have a round section of some length at the lower end, square spuds are subject to considerable torsion. Round spuds are free from this trouble. On the other hand, it is much easier to build a square steel spud than a round one. One steel spud was built 24 in. square from 1-in. plates and 8 by 8 by 1-in. angles. This spud was lagged to a round shape with hard wood; it was used in a 36-in. round spud well, and gave complete satisfaction.

The use of two steel spuds is open to one objection. Sooner or later the leverman will get careless and swing with both spuds down. When this happens it is preferable to have one wooden spud to replace rather than to have to deal with two bent steel ones. Even if the leverman never makes a mistake, a swinging wire has been known to carry away when the dredger was in a cross-current with both spuds down. One spud is used only to "fleet ahead", and there is no necessity for having this one as strong as the digging spud.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

HIGHWAY LOCATION: GENERAL CONSIDERATIONS

Discussion

BY MESSRS. C. S. MULLEN, AND H. L. THACKWELL

C. S. MULLEN,[4] ESQ. (by letter).[4a]—The author has covered his subject so thoroughly that a discussion is practically limited to an approval or disapproval of his statements.

Greater emphasis might be placed on the fundamental assertion that the life of a highway is not a function of surface, but of location. Thousands of examples all over the United States could be cited, where road surfaces entirely adequate to carry existing traffic for many years are being abandoned, because the location on which they were built is no longer safe or economical.

The use of aerial photography as an aid to highway location is comparatively new, but in many cases will prove very economical and at times invaluable.

The suggestion as to introduction of horizontal curves to secure increased sight distance is, in the writer's opinion, questionable. As the added sight distance is desirable for greater safety it is suggested that this safety feature could be better secured by widening the surface at summits, rather than by breaking up the alignment and introducing a curve which is a constant potential danger point.

Mr. Gilchrist's belief in the democratic principle that the majority should rule, is not a safe guide in highway location. It has been demonstrated a number of times that where this principle has been followed, serious mistakes have been made which later were condemned by that selfsame majority.

The question as to locating a highway so as to pass through all cities and small towns on its approximate route, is one of the most puzzling that confronts the highway engineer. It is believed that in such cases the question of traffic congestion should receive most serious consideration and the location made, if possible, to avoid this condition.

In conclusion, the writer believes Mr. Gilchrist's paper to be fundamentally sound, clearly expressed, and a valuable addition to the subject of highway location.

NOTE.—This paper by Gibb Gilchrist, M. Am. Soc. C. E., was presented at the Joint Meeting of the Highway and Construction Divisions, Dallas, Tex., April 25, 1929, and published in November, 1930, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[4] Chf. Engr., Dept. of Highways, Richmond, Va.

[4a] Received by the Secretary, December 24, 1930.

H. L. THACKWELL,[5] ASSOC. M. AM. SOC. C. E. (by letter).[5a]—The answer given to Mr. Gilchrist's questionnaire, that "the life of any particular type of road depends, first, on its location and, second, on the width of its right of way, and that the type of surface was of secondary consideration", is certainly true in part. There are other factors, however, that play an important rôle, namely, climate, attention to maintenance, drainage, etc. Highway maintenance funds should vary with the topography, traffic, and climate.

State highways in Texas, as located in 1930, certainly have been improved in alignment, and distances have been shortened by curve elimination. However, grades are still prominent by their frequent reversals. The rigid policy of exactly proportioning cut and fill, and not allowing waste fills or excessive borrows, is not in accord with the vision of to-morrow.

Many highways from 5 to 10 years old are being abandoned or their locations are being changed radically; such change of mind or policy would indicate that still further changes or eliminations may be expected in the near future on newer locations.

A city of several thousand people in Texas surely has an expectancy of life equal to 1 000 years. It is also reasonable to conclude, in spite of air travel, that wheeled traffic will still be in vogue 1 000 years hence. If these two assumptions are correct, the roadbed (earth foundation which has an exceedingly long life) should be designed and located so that there would be no likelihood of changes in grade or curvature for many decades in the future. Hard surfacing which may have an expected life of from 2 to 30 years (depending on the type, location, and traffic), may be changed in a few years without-violating fundamentally economic laws. Roadbeds, with right of way and drainage structures, often cost 50% of the total in places where a standard concrete pavement is used; hence, a mile of roadbed costing $20 000, that has an ultimate life of 1 000 years, would capitalize without interest at $20 per mile per year. The concrete surface might also cost $20 000 ($800 per mile per year), and have a life of 25 years.

The roadbed is frequently altered or abandoned by change of policy, administration, or vision, while it still may retain 99% of its expected life, but not its economic utility. The surface, however, is never abandoned or changed, until its useful life has been largely exhausted.

It does not take great imagination to visualize railroads replacing steel and ties with concrete surfacing and using huge wheeled vans drawn by tractors or individually propelled cars. They may run on their own roadbed in the country districts and, traversing cities, run on city paving and unload the cargo and freight at warehouses wherever they may be situated.

State highways of the future will abandon the present system of numerous short reverse grades and deep drainage ditches, and will incorporate storm sewers, curb, and gutters, with widened pavement, somewhat similar to city pavements, thus taking care of the ever-increasing traffic. Such a highway will provide relief from congestion, will increase safety, and will utilize the acquired rights of way economically.

[5] Cons. and Constr. Engr., Jacksonville, Tex.

[5a] Received by the Secretary, December 31, 1930.

The writer is of the opinion that aerial highway location should be followed by plane-table topography, the latter being conspicuous by its absence in Texas highway surveys.

Formerly, the improved highway locations for King County, Washington, were always projected on topographic maps which were sufficiently accurate to make possible an economic paper location that would conform to any preconceived rule as to alignment and grade economies.

The dissimilarity between highway location and city street location is marked most strongly by the tendency for highway grades to be above the adjoining property so that free drainage may be adequately maintained. In city paving, on the contrary, the grades are lower than abutting property, and storm water falls over the curbs and is ejected through the storm sewers. This cardinal difference in tendency has caused many cases of objectionable grade transition on roads located through small cities, by highway engineers, and of paving construction carried out by city engineers at a later date.

In conclusion, it is the writer's commentary, but not criticism, that it would be better to have fewer reversed grades, fewer deep ditches, greater use made of storm sewers and gutters, and a gasoline tax so proportioned for maintenance that the hilly country with high precipitation would have more maintenance funds per mile of road than the flat dry country of equal traffic weight. No State aid for cities should be permitted and State highways should begin and end abruptly at city limits. Cities should be required by State law to keep up an economic paved roadway connecting the State highway through the city, that is financed by refund of a proportion of the gas tax to the municipality. State highways should be financed, owned, and operated by the State and Federal Governments and these groups should not expect aid from counties or local communities. The so-called co-operation of funds is largely responsible for frequent criticism by local communities against the removal of existing highways, and has naturally culminated in selfish interference, through politics, with economic engineering locations. The engineer's call of distress is well justified, but the cure will be at hand only when the financing policies have been changed.

The grand success of the Appian Way and its magnificent permanence are due more to the splendid Roman administration and policy than to the engineering of the highway itself. The original location is unchanged and the foundation still endures, while gravestones crumble and empires fall.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

HIGHWAY LOCATION: PRACTICAL CONSIDERATIONS

Discussion

BY H. K. BISHOP, M. AM. SOC. C. E.

H. K. BISHOP,[2] M. AM. SOC. C. E. (by letter).[2a]—The primary use of a modern highway, generally speaking, is to provide means for vehicular transportation of the various classes of travel and commercial haulage from one common center to another, either local, State, or National. They should be established over the most direct route possible with the least gradient. The most direct route may be defined as a road with alignment free from curves that have dangerously short sight-distances based on the importance of the road and the speed limit involved; and the least gradient should be such that it will not retard materially the reasonable speed of present-day vehicles.

The most direct route and least gradient, however, may be dispensed with advantageously, to some extent, in certain areas of heavy snowfall for reasons of economy in snow control. These factors may be ignored also in cases of scenic development, when locating forest and park roads or other lanes of traffic promoted solely for pleasant landscape effect, recreation, or educational travel.

For the winter of 1929-30 the States in the heavy snowfall area of the country reported that about 185 000 miles of main rural roads were kept clear of snow at an approximate cost of $8 250 000, and statistics show that more than 779 000 private automobiles entered, and more than 2 270 000 people visited the National parks during the 1930 season. Also, more than 28 760 000 people visited the various National forests by automobile during 1929.

The cost of clearing roads of snow in areas presenting certain topographical features can be greatly reduced by appropriate study of the proposed locations, and consideration should be given to the many patrons of the great out-of-door sections by creating alignment for the roads built, that will unfold, to the best advantage, the scenic wonders of the region visited.

Location to Prevent Snow Drifting.—Removal of deep snow is likely to create considerable expense in regions subject to heavy snowfall, and anything

NOTE.—This paper by A. R. Losh, Assoc. M. Am. Soc. C. E., was presented at the Joint Meeting of the Highway and Construction Divisions, Dallas, Tex., April 25, 1929, and published in November, 1930, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[2] Chf., Div. of Constr., U. S. Bureau of Public Roads, Washington, D. C.

[2a] Received by the Secretary, January 3, 1931.

that can be accomplished to prevent drifting is of great value, not only from the viewpoint of economy, but also for the convenience of winter traffic. Snow drifting is often prevented or greatly reduced by proper road location. For this reason a number of highway departments give the snow problem careful consideration in planning their new or relocated roads, and, consequently, they experience fewer blockades on highways recently constructed than on the older ones. For drift prevention, proper location is often of primary importance and is influenced by efforts to keep the roadbed away from northern slopes and to eliminate various other conditions that tend to cause drifting.

The locating engineer for roads of this nature should be familiar not only with the territory in which he is engaged in connection with ordinary features, but he also should have a knowledge of snowfall and prevailing wind conditions. In some instances, the sacrifice of alignment and grade, to a reasonable extent, may be necessary to secure drift-free roads. In open areas the highway should follow ridges where possible or should be placed on slopes against the prevailing winter winds. Long shallow cuts should be avoided, but where this is impracticable the location and construction should be such that they will not cause drifting.

In low flat country snow drifting has been lessened and often eliminated over a location where the new grade can be elevated 2 or 3 ft. above the surrounding land and with the construction of 1 on 4 slopes. In instances of this kind it would also be better to widen the cuts on either side to the boundary lines of the road. In mountainous or sparsely settled areas where the winter traffic is not sufficient to justify the expense of keeping the roads open during the entire winter, proper exposure is highly essential in order that the snow may melt and free the road for traffic as early in the spring as possible.

In States with heavy snowfall a winter reconnaissance should be made, or at least an inspection, during the snow season and prior to the completion of the plans in order that all snowfall, wind, and drifting conditions may be given careful consideration and data submitted for the use of the locating engineer in the final determination of his alignment. A number of these and other methods have been followed by various States and, in some instances, have caused the roads to remain fairly free of drifts through the winter months.

Tourist Parkways.—In the location of roads proposed to carry ordinary traffic, examination of the country through which it is to pass, with reference to the topographical features encountered, is necessary. Furthermore, consideration must be given to the nature and extent of traffic. On the other hand, for sight-seeing roads, other questions may arise regarding the proposed lines relative to greater comfort and convenience of travel and also regarding the pleasure and knowledge to be derived from use of the roads proposed. Often esthetic and historical features are present that suggest modifications of alignment that would not appear as good engineering practice in choosing locations for common roads.

In locating State and county roads, existing traveled ways are often utilized with necessary changes here and there to improve the alignment and grades; but for park and forest roads it is at times desirable to avoid all traveled thoroughfares, both in regard to following their alignment and in

crossing them. Recreational roads are built principally for sight-seeing traffic and should be located with the convenience of the traveler in mind regardless of reasonable extra expense. Where intersections of existing roads are unavoidable, under or over-passes should be provided with ramps constructed where necessary for feeders to the parkway.

For ordinary roads the most suitable locations are those that reduce the yearly expense for transportation to the minimum, reduce the annual maintenance cost for the roadbed, and keep the original cost of construction as low as possible consistent with good engineering practice. These elementary problems are also applicable to tourist roads in greater or less extent, especially relative to the cost of construction. This first cost depends largely upon adjustment of the grade to the existing surface in order that minimum quantities may be involved in excavation and embankment and the location determined to avoid in the largest degree both major and minor drainage structures. For all types of roads great care must be exercised in selecting locations away from unstable foundations, such as swamps and soft ground, where construction is expensive and drainage difficult.

In locating tourist roads existing historical sites must be skirted to provide interest for the traveling public, but they must not be mutilated in any way by proximity or intersection. While wooded routes are generally desirable, large or rare trees must be spared and saved from the axe. In building parkways deep cuts are apt to cause scars in the natural landscape and, therefore, are not allowed. In places where embankments are essential, instead of balancing quantities in the usual manner, a reasonable cut is made with the remaining material provided by borrow from pits out of sight of the traveling public. When rock cuts and fills are unavoidable their new surfaces are re-covered with top soil for vegetation growth. Consequently, in locating roads of this character, the availability of suitable loam to mask the scars inflicted must be considered.

Forest and park roads usually traverse one or the other of these playgrounds, form a parkway between them, or act as lanes connecting historical points. Their routes vary in topography from flat and rolling country at tidewater levels to rough and rugged mountainous regions 10 000 ft. and more above sea level. Such proposed roads, however, usually extend through scenic country primarily for recreational purposes. From their nature they follow wooded and wild territory where location is difficult. At times, in fact, the degrees of roughness encountered are such that tunneling is considered more economical and desirable. For these classes of roads, as well as for most others, study of the proposed alignment should begin with the reconnaissance and should continue until the improvement is advertised or construction actually started.

For wooded sections or regions with irregular configuration, the benefit of aerial photography can not be over-estimated. This is especially true considering the fact that a reconnaissance should embrace a territory and should not merely follow a line. From these photographs a strip of land for location can be approximated between control points. In a general way the proposed location can be narrowed thus within reasonable limits. With this accomplished, topographical maps in one form or another are very useful. Drawn to a scale of 1 : 62 000 such maps are available for most sections of

the country. While these are helpful in determining a line, maps covering the same territory greatly enlarged are procurable from the various Federal agencies at least at reasonable cost.

With maps covering the territory selected from the aerial photographs enlarged to about 400 ft. per in., the best and most economical alignment can be determined with surprising accuracy. Another method followed by the Government is to use maps for approximate alignment data compiled from so-called strip, topography surveys. The work is performed by the U. S. Geological Survey at a cost per square mile depending upon the scale desired. These strip, topography maps are prepared in about the same manner and carry data similar to those found on the ordinary U. S. Geological Survey maps, but the area is confined to the strip selected—probably $\frac{1}{8}$ or $\frac{1}{4}$ mile wide—with the salient points located, if possible, with greater care.

With Geological Survey maps at hand, especially those enlarged, or others plotted from strip, photography surveys with accurate contours, a trained and competent location engineer will note the direction of flow of streams; the positions of thick or light wooded sections; the height and formation of the various slopes; the position of ridges that may be followed; and the location and elevation of the various passes through which it might be possible to carry the line; and from them he will adopt the most suitable alignment.

Scenic routes generally are proposed for country composed of numerous valleys enclosed by ridges of varying heights, all branching in like direction with the main watercourse of the territory. With a careful study of these principal features from the map used, the location may be determined with a fair degree of accuracy, leaving only the minor details to be settled for final location and for staking out the center line.

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DISCUSSIONS

FLOW OF WATER IN TIDAL CANALS

Discussion

BY R. L. FARIS, M. AM. SOC. C. E.

R. L. FARIS,[23] M. AM. SOC. C. E. (by letter).[23a]—This paper is an important contribution to a subject that is beset with many complexities. Throughout the world, and more especially in the United States, the subject of tides has been developed very largely by the mathematician whose interest, primarily, was in other than the engineering phases of the subject. It is gratifying, therefore, to find the fundamental problems of the subject successfully attacked from the engineering point of view.

In introducing Table 3, under the heading, "Heights", the author states that,

"It is the practice of the United States Coast and Geodetic Survey to allow for lack of symmetry between positive and negative waves in making tidal predictions by introducing a 'shallow-water' component into the computations."

This may leave an erroneous impression with the reader not familiar with the problem of the prediction of tides, and it appears desirable to amplify the subject somewhat.

In connection with the prediction of tides it is unnecessary to assume the existence either of positive or negative waves in the sense used by the author. The problem centers in the resolution of the tides, as actually observed at any given place, into a number of simple cosine curves each having a definite period, amplitudes, and phase. As soon as these become known, the tide for any future time can be predicted by combining these simple cosine curves for the time in question. Technically, each of these simple curves is spoken of as a component of the tide.

The periods of the primary components of the tide are derived from astronomic considerations, and, on the open coast, the tide as actually observed can be closely represented by the combination of a number of such components. However, in traversing the shallow waters of tidal rivers, or other like bodies of water, the open ocean tide becomes distorted or modified.

NOTE.—The paper by Earl I. Brown, M. Am. Soc. C. E., was published in December, 1930, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[23] Asst. Director, U. S. Coast and Geodetic Survey, Washington, D. C.

[23a] Received by the Secretary, January 14, 1931.

The modification that the tide undergoes in its progress up a tidal river is illustrated in Fig. 21 which represents the rise and fall of the tide at Fort Hamilton, New York, and Albany, N. Y., for the 2-day period, May 20 and 21, 1914. The lower curve is the tide curve for Fort Hamilton, which may be regarded as situated near the mouth of the Hudson River, while the upper curve is the tide curve for Albany, near the head of tide-water on the Hudson, about 130 nautical miles above Fort Hamilton.

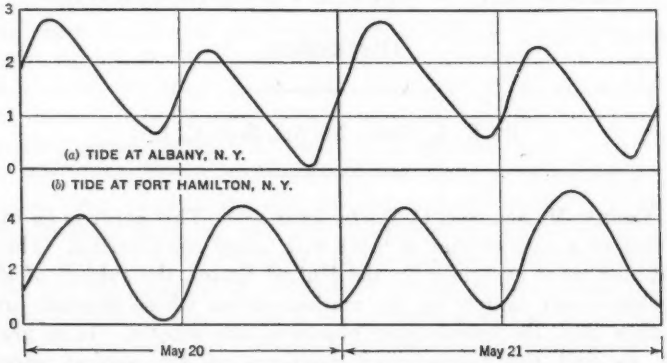

FIG. 21.—TIDE CURVES AT ALBANY, N. Y., AND FORT HAMILTON, NEW YORK, FOR MAY 20 AND 21, 1914.

The tide curve for Fort Hamilton is representative of the tide on the open coast in that the durations of rise and fall are approximately equal. This is evidenced by the symmetry of the curve. At Albany, however, the tide has become modified considerably. In overcoming the friction against its progress up stream, the duration of rise has become shortened, giving a corresponding increase to the duration of fall. In Fig. 21 this is represented by the steepness of the rise of the tide which contrasts markedly with the much gentler slope of the fall.

In resolving river tides into their constituent simple tides for the purposes of prediction, the so-called shallow-water components come to light, quite apart from the causes which bring them about. In this connection, therefore, it is better not to speak of "introducing" shallow-water components, because these exist in river tides and must be taken into account in the prediction of river tides.

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DISCUSSIONS

SILTING AND LIFE OF SOUTHWESTERN RESERVOIRS

Discussion

BY T. U. TAYLOR, M. AM. SOC. C. E.

T. U. TAYLOR,[1] M. AM. SOC. C. E. (by letter).[1a]— In describing the silting condition of the Colorado River in Texas the author mentions some observations reported by the writer in 1928.[2] Since that time an aerial photographic map has been made of the dam and the lake at Austin. It covers a stretch of the river 4 000 ft. long, including Bee Creek which is a tributary from the west. Fig. 1 is a sketch drawn with the assistance of that map. The original water surface of 1913 includes all the area bounded by the solid lines. The present channel is confined between the dotted lines when the water is level with the crest of the new dam at Elevation 51.4. The part between the solid lines and the dotted lines shows the accumulation of silt. It indicates that the silt is gradually narrowing the channel and has reduced it from a width of 1 300 ft. in 1913 to 620 ft. in 1929, with the Bee Creek cross-section ½ mile up stream. The area on the right has been reclaimed entirely and consists of about 14.6 acres of silt.

There are several striking points illustrated by this survey. Point *A* (Fig. 1) is in the original river bed where the water in 1913 was 51.4 ft. deep. This depth has all been taken and the space is now a sand beach covered with willows and weeds.

At Point *B*, in the mouth of Bee Creek, the water was originally 51 ft. depth in 1913. It has now a bar of silt 2.4 ft. above the lake level, making a depth of 53.4 ft. of silt.

In 1913, a boat-house was anchored at the sharp point, *K*, near the word, "Bee", and a camping party dived off the boat into 30 ft. of water. In 1930 it has become "high and dry," and Bee Creek from head to mouth is a solid plug of silt varying in depth from 53.4 ft. to zero. On the west shore, a strip of land has been reclaimed or taken from the lake by the encroachment of the silt that extends 5.2 miles up the lake, varying in width from 100 to 300 ft.

NOTE.—The paper by the late R. G. Hemphill, Assoc. M. Am. Soc. C. E., was presented at the meeting of the Irrigation Division, Dallas, Tex., April 25, 1929, and published in May, 1930, *Proceedings*.

[1] Dean of Eng., Univ. of Texas, Austin, Tex.

[1a] Received by the Secretary, May 10, 1930.

[2] "The Silting of the Lake at Austin," Univ. of Texas, *Bulletin No. 2439*.

Measurements made opposite the mouth of Bee Creek in 1929 revealed a cross-section of 2 400 sq. ft. as compared with 37 300 sq. ft. in 1913. It is thought that the 1929 maximum depth opposite Bee Creek will scarcely be

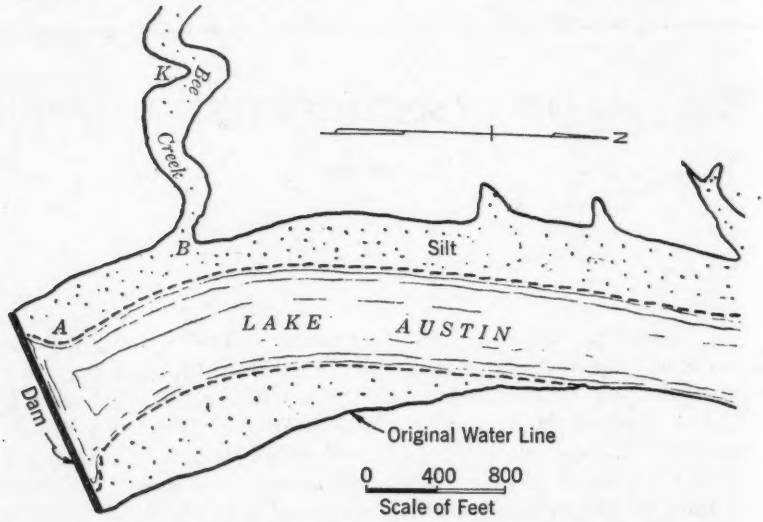

FIG. 1.—SKETCH SHOWING EXTENT OF SILT ACCUMULATION.

decreased. Other phases of this subject were discussed more fully in connection with another paper.[3]

[3] "Silting of the Lake at Austin, Texas," by T. U. Taylor, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1681.